WATER RESOURCES DEVELOPMENT PROJECT

CHARLES RIVER DAM

CHARLES RIVER BASIN, MASSACHUSETTS

DESIGN MEMORANDUM NO. 4

EMBANKMENTS AND FOUNDATIONS

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS.

FEBRUARY 1972

WATER RESOURCES DEVELOPMENT PROJECT

CHARLES RIVER DAM
CHARLES RIVER BASIN
MASSACHUSETTS

Design Memoranda Index

No.	Title	Anticipated Submission Date	Date Submitted	Date Approved
1	Hydrology and Tidal Hydraulics		21 May 71	2 Aug 71
2	General Design, Site Geology and Relocations		14 Feb 72	
3	Concrete Materials		19 Feb 71	29 Mar 71
4	Embankments and Foundations		22 Feb 72	
5	Pumping Station	Mar 72		
6	Vehicular Viaduct	Feb 72		
7	Navigation Locks and Facilities	Mar 72		
8	Cofferdams	May 72		

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WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER DAM
CHARLES RIVER BASIN, MASSACHUSETTS

PERTINENT DATA

Purposes	Flood Control, navigation, and highway transportation.
Location	
State	Massachusetts
County	Suffolk
City	Boston
River	On the Charles River 2,250 feet downstream of the present Charles River Dam.
Drainage Area	
Total Watershed	307 square miles
Lower Charles River above proposed dam	58 square miles
Surface Area	
Datum Relationship	M.D.C. Base is 105.65 feet below mean sea level (MSL) U.S.C. & G.S. Datum of 1929 [105.65 M.D.C. = 0.0 MSL U.S.C. & G.S. (1929)]
Proposed Basin	705 acres
Tidal and Flood Conditions	
Design High Water (Tidal)	Elevation 113 M.D.C. Datum
Design Low Water (Tidal)	Elevation 100* M.D.C. Datum
Normal Basin Level	Elevation 108 M.D.C. Datum
Maximum Desired Basin Level	Elevation 110 M.D.C. Datum
Basin (Prelowering) Level	Elevation 106.5 M.D.C. Datum

* For embankment considerations only.

Embankment	
Maximum Height above River Bottom	35 feet
Total Length	630 feet
Bearing Capacity	
Maximum Allowable Bearing Pressure	4 T.S.F.

PART A --- EMBANKMENTS

A. INTRODUCTION

1. General

a. Description. The Charles River Dam Project will include earth embankments connecting the concrete structures within the Charles River channel to higher ground on both sides. The primary function of the embankments will be to act as a dam between the fluctuating harbor tides on the downstream side and the relatively constant Charles River Basin upstream. The site plan is presented on Plate 4-1. In all cases, elevations referred to in this Design Memorandum are based on the M.D.C. Datum (Elevation 105.65 equals Mean Sea Level).

b. Purpose. This memorandum presents the results of subsurface invesitigations and soil engineering studies undertaken for the design of the Charles River Locks and Dam. The subsurface investigations include programs of subsurface explorations and laboratory tests, conducted to determine the distribution and characteristics of foundation materials and to determine soil conditions pertinent to excavations and to the design and the construction of embankments. Soil engineering studies, based on data obtained from the subsurface investigations, were conducted to develop safe and economical earthwork designs and construction methods.

B. SUBSURFACE INVESTIGATIONS

2. Subsurface Explorations. Subsurface explorations were initially made at this site in 1962 for the Metropolitan District Commission. These explorations consisted of drive sample and rock core borings, and were made using techniques and operations commonly accepted as local practice in the Boston area. Soil samples were obtained by using a 2 inch O.D. (1 3/8 inch I.D.)

split-spoon sampler driven 18 inches by a 140 pound hammer free-falling 30 inches (i.e., the "standard penetration resistance test").

In order to update this information, additional borings were taken by the New England Division, Army Corps of Engineers in 1970 and 1971. These subsurface explorations were programmed and made in conformance with the current criteria and practices as described in Corps of Engineers Manuals EM 1110-2-1801 "Geological Investigations" and EM 1110-2-1803, "Subsurface Investigations-Soils". The majority of these explorations were drive sample borings. There was generally good agreement between the results of the two subsurface exploration programs. The total results of the subsurface exploration programs completed to date are considered adequate for design purposes. The locations, types and general purposes of the explorations, as well as the geology of both the site and the area which is pertinent to the types and distribution of soils, are described in Design Memorandum No. 2, "General Design, Site Geology and Relocations".

3. Laboratory Tests. All laboratory tests were performed in accordance with current standard procedures as described in the Corps of Engineers Manual EM 1110-2-1906, "Laboratory Soils Testing". Soil samples from the recent subsurface exploration program were visually classified in the laboratory in conformance with the Unified Classification System. These visual classifications were confirmed by grain size analyses and Atterberg Limit determinations on samples considered to be representative of the various soil types encountered. Additional tests were performed on selected samples to determine their natural moisture content, natural density, consolidation and shear strength characteristics.

4. Presentation of Data. The results of the subsurface explorations accomplished by the New England Division, Army Corps of Engineers are presented in this memorandum, except that the geologic sections are included in Design Memorandum No. 2, "General Design, Site Geology and Relocations". A summary of the results of laboratory tests is presented in Appendix A. Logs of undisturbed samples, gradation curves and detailed shear test data reports for organic silts from exploration FD-IA are presented in Appendix B. Detailed consolidation test data for these materials is included in Appendix C. Selected test data for till are shown on Plate 4-14. The locations of all borings drilled in 1962 and 1970-71 are shown on Plate 4-2. The logs of subsurface explorations are presented in Plates 4-3 through 4-12. Although many borings are in areas discussed more fully in Part B, Foundations, of this

memorandum, they are grouped together for convenience. Generalized soil profiles along selected sections are shown on Plate 4-13.

C. CHARACTERISTICS OF EMBANKMENT FOUNDATION SOILS

5 . General Profile. The generalized soil profile along the baseline of the vehicular viaduct, including the Boston and Charlestown embankments, is shown on Plate 4-13, together with the embankment limits and structure locations.

a. Boston Embankment. The existing level ground surface comprising the Boston shore is a result of the gradual filling of the river-front with granular and miscellaneous fill for commerical development. This fill was substantially completed more than 100 years ago. It extends from a point south of Causeway Street, where it meets higher natural ground, a distance of over 1000 feet to the present river edge, and averages about 30 feet in thickness. In many areas the fill is capped by a thin layer of pavement, rip-rap, railroad ballast or other materials. The Warren Avenue Bridge, a pile-supported timber structure, connected the Boston and Charlestown shores until it was destroyed by fire and subsequently abandoned in the mid-1950's. No effort has yet been made to remove the structural remains, nor its pavement and associated utilities. Underlying the granular fill is a continuous layer of sandy to clayey organic silt, and organic siltly clay referred to hereinafter as organic silt. It varies in thickness from about six feet near the river edge to 15 feet some 300 feet away from the river. In the river area the layer of granular fill is absent. A deposit of silty sands lies under the organic silt, and varies in thickness from 7 to 15 feet. A layer of till is next encountered, and its thickness decreases from about 30 feet near the river edge to 15 feet at the southerly end of the project. This layer is, in turn, underlain by the soft Argillite bedrock, with a surface at about Elevation 50. The observed water level in the bore holes was influenced by tidal fluctuations, and varied between the mean high and mean low water levels.

b. Charlestown Embankment. The same general site-filling conditions and soil profile found at the Boston side also exist at the Charlestown embankment, the major difference being the absence of the organic silt layer on the Charlestown side. There is a surface layer of about 15 feet of fill overlying 6 feet of silty sand and underlain, in turn, by 35 feet of till. The layer of granular fill is not present in the area within the river. The Argillite bedrock is at a depth of 60 feet below the present ground level, or about Elevation 60. The observed water levels ranged from mean low to mean high water, and were influenced by tidal fluctuations.

6. Soil Properties.

a. Existing Fill. The surface fill is classified as SP-SM and GP-GM, and consists of a mixture of silts, sands, and gravels together with ashes, bricks and cinders. The moist unit weight is estimated to be 110 pounds per cubic foot. For purposes of design, the angle of internal friction is considered to be 33 degrees.

b. Organic Silt. The soft, dark gray to black organic silt contains varying amounts of shells, wood fragments and fibrous material. There are also random pockets and lenses of fine sand, and occasional pebbles. Laboratory tests were conducted on 3 inch undisturbed stationary piston samples of the organic silt obtained during the subsurface investigation program conducted by the Corps of Engineers in 1970-71. The laboratory test program consisted of three basic test categories, with the following results:

(1). Classification Tests. Included in this category were such tests as Atterberg Limits, Visual Classifications, Unit Weights, Organic Content Determinations, Specific Gravity and Hydrometer Analyses. The summaries of these test results are presented in Appendix A of this memorandum. The tests accomplished on the samples of Organic Silt (OH) soil indicate that the specific gravity is between 2.52 and 2.60. The organic content ranges from 6.0 to 9.0 percent. Total unit weights vary from 93 to 97 pounds per cubic foot (pcf), and for design purposes the total unit weight was adopted as 95 pcf. The Liquid and Plastic Limits vary from 54 to 109 percent and 28 to 42 percent, respectively, thus giving a range of the Plasticity Index between 26 and 67 percent. Natural water contents based on dry weights range from 53 to 80 percent. Between 26 and 32 percent by weight of this material has a particle size less than 0.002 mm.

(2). Compressibility Tests. Two consolidation tests were accomplished. A fixed-ring consolidometer apparatus was used in all tests, with a ring having dimensions of 4.44 inches diameter and 1.0 inch high. The final pressure to which the samples were subjected was 8 tons per square foot.

The durations of both the loading increment and the unloading decrement were twenty-four hours for all tests. The coefficient of consolidation (Cv) was determined from the curves of time (plotted on a logarithmic scale) versus dial readings (arithmetic scale) for each load increment in each consolidation test. The load-compression (e-log p) curves for each test, are presented in Appendix C. The test results for the Organic Silt show that the compression ratio ($\frac{Cc}{1+e_o}$) is between 0.22 and 0.24, and that the material is normally consolidated. The coefficient of compression (Cc) ranges from 0.65 to 0.74. The coefficient of consolidation (Cv) ranges from 4 to 13 x 10^{-4} cm^2/sec., and the initial void ratio (e_o) varies from 1.92 to 2.05.

(3). Triaxial Compression Strength Tests. Two series of consolidated-undrained (R) tests were performed on the organic silt. The tests were run applying back pressure and recording pore pressure measurements. Two series of unconsolidated-undrained (Q) tests were also run. Test data and sketches of samples at failure are included in Appendix B. In order to obtain the Mohr strength parameters for slope-stability analyses, Mohr Circle plots of shear stress vs. normal stress were prepared using the test results. These plots were drawn for both the total and effective stress conditions, and are shown in Appendix B. The Mohr strength parameters and the values developed for use in design are as follows:

Strength from R Tests

C = 0.10 to 0.14 tsf	Use 0.10 tsf
ϕ = 13.5 to 13.9°	Use 14°

Strength from Q Tests

C = 0.22 to 0.27 tsf	Use 0.24 tsf

c. Silty Sand. This material is mainly classified as a silty sand (SM). It consists of predominantly gray to black silty sand with occasional shells and cobbles or silty sandy gravel (GP-GM). In some locations it has a slight organic odor.

d. Till. This material ranges from sandy silty clay (CL) to a sandy clayey gravel (GC) with cobbles. Samples of this material were well-graded, with from 40 to 70 percent by weight finer than the No. 40 sieve. The dry unit weight varies between 130 and 145 pcf. The natural water content based on dry weight varies from 6 to 17 percent, and is generally between 8 and 12 percent. The in-situ material was found to be so dense during the exploration phase that conventional drive-sampling procedures could not always be used. Core-drilling was frequently resorted to in order to obtain representative samples of the till. Selected test data for the till is shown on Plate 4-14.

7. Bedrock Properties. The bedrock at the site consists of Cambridge Argillite. Because of its relative depth and the type and thickness of the soil overburden, it is not of engineering significance for purposes of embankment design and construction. However, the bedrock is of significance for foundation design of structures, especially where piles are used. A more detailed description of the bedrock is included in Design Memorandum No. 2, "General Design, Site Geology and Relocations."

D. DESIGN CRITERIA

8. Tidal and Flood Conditions. Design Memorandum No. 1, "Hydrology and Tidal Hydraulics," describes the background data and reasoning by which were established the theoretical tidal and flood conditions which might occur at the Charles River Dam, if the several contributing factors were to be acting simultaneously. The criteria which effect the design of the embankment are:

a. Design High Water	Elevation 113.0(MDC Datum)
b. Design Low Water	100.0*
c. Normal Basin Level	108.0
d. Maximum Desired Basin Level	110.0
e. Basin (Prelowering) Level	107.0
f. Maximum Differential Head (Tide Side to Basin)	6.0
g. Significant Wave Heights	
(1) Basin	1.0
(2) Bay Side	2.9

*This level is for embankment considerations. The design low water level for hydraulic considerations is Elevation 102.5.

9. Stone Protection. Stone sizes required for protection of the embankment slopes were determined in accordance with Corps of Engineer's Manual EM 1110 - 2 - 2904, "Design of Breakwaters and Jetties."

E. DESCRIPTION OF EMBANKMENT

10. General. The two embankment sections connecting the river structures to the Boston and the Charlestown shores will consist of granular materials (a reasonably well-graded sandy gravel or a gravelly sand). Embankment fill materials will be provided by the Contractor from off-site sources. Typical sections are shown on Plates 4-21 and 4-22. The top of the embankment will generally be at Elevation 118, which also corresponds to the top of the boat lock structures. The side slopes will be 3 horizontal to 1 vertical, and will be protected by stone. Where required for aesthetics, square-cut quarried stone will be used in the above-water portions.

11. Embankment Sections

a. Compacted Gravel Fill. For the embankment constructed in the dry, the embankment section was selected on the basis of the minimum side slope which would satisfy stability requirements. However, since the portion of embankment constructed in the dry is limited, the effects of the design details of adjacent sections constructed in the wet and the existing slopes to remain govern the embankment shape.

b. Dumped Gravel Fill. A substantial portion of the embankment must be constructed in the wet, beyond the limits of a cofferdammed area. Gravel fill will be dumped underwater below Elevation 105 without compaction.

12. Slope Protection. Both the tidal and the basin slopes of the embankment will be provided with a surface layer of stone protection, placed upon stone bedding. The stone protection will be transitioned from the embankment slopes to the existing conditions along the Boston and Charlestown shores. The design of the protection stone is based on anticipated velocities and/or wave considerations, following the criteria and practices as described earlier. Because of the relatively small quantities of protection stone required for the overall project, it was decided for economic reasons to specify a minimum number of composite stone gradations rather than one for each small area to be protected. The selected

gradation for the range of anticipated velocities and wave conditions, from 5 to 300 lbs. will satisfy these criteria. Because of the location of this project and its important visual exposure to the community, the faces of the embankment will be provided with square-cut stone protection extending from below the lowest water level anticipated to the top of the slope. The exposed face of this stone is about 2 feet by 4 feet, and its thickness will be about 1 1/2 to 2 feet. This protection will be more than adequate in satisfying the criteria against the anticipated wave action (for waves up to the expected maximum of 3 feet) and against ice scour action due to the relatively large tide range on the bay side.

F. CHARACTERISTICS OF EMBANKMENT MATERIALS

13. Materials.

a. Gravel Fill.

(1) General. Except where otherwise shown for specific purposes, all earth fills and backfills are to be of granular materials consisting of reasonably well-graded sandy gravel or gravelly sand. These granular materials may be any natural or processed bank-run material, provided that they are reasonably free from thin, flat and elongated pieces and contain no soft or friable particles.

(2) Segregation. All loading, hauling, handling and stock-piling operations shall be performed in a manner that will prevent segregation and will permit placement of well-graded materials.

(3) Gradation. Gravel shall be well-graded throughout the entire range of particle sizes, and shall be graded so as to meet the following requirements:

U.S. Standard Sieve Designation	Percent Passing, by Weight
6 inch	100
2 inch	75-100
1 inch	50-85
No. 4	40-70
No. 40	20-50
No. 200	0-8

(4) Sources. There are no materials to be excavated and removed from the project excavations which are anticipated to meet these requirements. Investigations of the probable sources of gravel indicate that similar gradation specifications have in the past been satisfied by materials available from commercial sources. Locations of particular sources which might be available depend upon many currently unknown factors but the most likely sources are within 75 miles.

b. Protection Stone and Bedding Stone.

(1) General. All materials required for rock slope protection shall be from off-site sources. The material for bedding stone and protection stone shall be hard, durable, and sound rock, weighing not less than 160 pounds per cubic foot in its natural state.

(2) Bedding Stone. Bedding stone shall be sound crushed stone, well-graded in size from 1 inch to 3 inches. The material shall not contain more than 25%by weight of thin, flat and elongated pieces; and shall contain no organic matter, fines, nor soft friable particles.

(3) Type I and Type II Protection Stones. Protection stones shall be durable fragments of quarried or blasted rock and shall be well-graded from 5 to 300 pounds and from 50 to 500 pounds for types I and II, respectively. They shall contain no fragments having a maximum dimension greater than 2 feet and shall be free of organic matter and friable particles. Type II stone will be used only for protection of the channel against scour.

(4) Type III Protection Stone. Type III protection stone shall be composed of sound pieces of square-cut quarried granite, each shaped as nearly as practicable to a right rectangular prism. All quarried protection stone shall be supplied from an approved source which will be able to provide sufficient stone of similar color and texture for the entire project. Quarried protection stone shall be between 1 1/2 feet and 2 feet in thickness, and each individual stone shall weight at least 2000 pounds. Quarried protection stone may vary in dimension and weight only when necessary for placement adjacent to the roadway curbing and at other confined areas.

G. CONSTRUCTION CONSIDERATIONS

14. Construction Procedure.

a. General. The embankment will be constructed in sections, partly in the wet and partly in the dry, typical embankment sections are shown on Plates 4-21 and 4-22. The actual limits of embankment materials to be placed and compacted in the dry will be determined by the inboard limits of the cofferdammed area. Embankment materials beyond the cofferdam limits will be dumped in the wet to 4 feet above mean low water (Elevation 105), and the remainder placed when the tidal waters are below the levels being worked. Embankments shall be constructed up to Elevation 105 on both sides of a cofferdam before removing the cofferdam.

b. Embankments Constructed in the Dry.

(1) Foundation Treatment. The foundation areas upon which embankment fill is to be placed shall be excavated or stripped within cofferdammed areas to remove all mud, debris, trash, paving, lumber and other unsuitable materials. During the placement of all embankment fill, the foundation area shall be free of water and shall not be frozen.

(2) Construction.

(a) General. Embankment sections shall be constructed by placing and compacting the gravel in essentially horizontal layers. All temporary construction slopes at the ends of fills shall not be steeper than 4 horizontal to 1 vertical. The thickness of each layer before compaction with a tractor shall not be more than 8 inches, and shall not be greater than 3 inches when compacted with a power vibratory tamper. Special care shall be taken to insure a tight contact of the compacted fill against all concrete surfaces and with the faces of all steel sheet piling, taking particular care to avoid any damage to the concrete walls and steel sheet piling.

(b). Equipment. Except in confined areas and adjacent to concrete structures and steel sheet piling, each layer of gravel is to be compacted by a crawler-type tractor weighing not less than 35,000 pounds. The tractor shall exert a unit tread pressure of not less than 9 pounds per square inch and shall have standard width treads. Tractors shall not be used to compact fill within 4 feet of steel sheet piling. Power vibratory tampers shall be used for compaction of fill materials in confined areas and in areas adjacent to concrete structures where tractors are not permitted.

(c). Procedure. Each fill layer shall be compacted by not less than 6 complete passes of the crawler-type tractor. A complete pass shall consist of the entire coverage of the area with one trip of the roller overlapping the adjacent trip not less than 2 feet. In a confined area where a vibratory-type power tamper is used, each layer shall be compacted by not less than 6 coverages of the tamper.

c. Embankments Constructed in the Wet.

(1). Foundation Treatment. Prior to the underwater placement of gravel fill, materials that have sloughed or settled onto the foundation area must be removed. During foundation cleanup prior to placing gravel upon excavated slopes, equipment must drag the sloping area to insure that it is at the desired grades.

(2). Placing. Gravel shall be placed underwater to the outboard faces of the cofferdams and to the required lines, grades and slopes as soon as practicable after the foundation clean-up operations have been completed in any reach to be filled. The lower portions of the underwater fill (below Elevation 105) shall be placed by skip or bucket lowered into the water and opened only a few feet above the placing level. Placement by end-dumping and pushing from above-water shall be used when suitable for the upper portions of underwater fill. Fill shall be placed in such a manner that the entire section will be free from layers or pockets of organic silt or other foreign material. Filling shall start at one end of the reach and proceed in one direction to Elevation 105. Compaction will not be required for gravel placed underwater. After the underwater embankment materials are placed, the side slopes will be trimmed to the required lines and grades.

(3) Embankment Above Elevation 105. The portions of embankment outside cofferdammed areas and above Elevation 105 will be placed and compacted in the dry, sequenced so that the tidal waters are below the levels being worked. Subsequent tides may inundate the fill, and care must be taken to avoid any deterimental effects from this submergence within the completed embankment. Above the high water level, the embankment construction can be scheduled regardless of the tidal fluctuations.

d. Protection Stone.

(1) Temporary Stone Protection. Temporary stone protection is required during the Stage I construction phase to protect the newly-excavated bottom and side slopes of the by-pass channel from scour. The maximum

river-bottom velocities through the channel are estimated to be in the order of 8 fps, and Type II protection stone will satisfy these conditions.

(2) Bedding Stone. Bedding stone within the dewatered cofferdammed areas or above Elevation 100 shall be placed in the dry directly from trucks, bucket or skip. Bedding stone placed in the wet shall be placed by bucket or skip, lowered into the water and opened just above the desired level.

(3) Embankment Slope Protection Stone. Type I protection stone is to be placed below the low water levels in both the upstream (basin) and the downstream (bayside) slopes. By design wave criteria, the stone weight should be 20 pounds and 140 pounds on the upstream and downstream sides, respectively. To meet these and other project considerations, the Type I stone is to be graded from 5 to 300 pounds (i.e., about 4 to 18 inches in spherical diameter). Protection stone will be extended landward from the bayside and basin slopes of the embankment to the Charlestown and Boston shores.

(4) Square-cut Protection Stone. Type III protection stone is to be placed upon the exposed faces of both the upstream and downstream slopes (i.e., bayside and basin sides). The square-cut Type III protection stone will be laid in course on stone bedding.

H. SEEPAGE CONSIDERATIONS

15. Seepage Control

a. Seepage through Embankment. Seepage through the dam embankment will be relatively minor, due to the relatively small and reversible seepage gradients resulting from the relation of the basin and tide levels. With a "constant" basin level at Elevation 108, the differential heads are 5 feet (upstream) at design high water and 8 feet (downstream) at design low water. These differential heads reverse themselves in a period of about 6 hours, so that the classic "steady-state" condition is never achieved within the embankment. Horizontal flow paths through the compacted portion of the embankment at the maximum head differential exceed 130 feet. Based on these conditions, no special means are required to control seepage. The bedding course beneath the stone protection will provide filter action for that minor quantity of sea water which would seep into, and then out of, the downstream embankment face during the tidal cycle.

b. Seepage through Foundation. The major portions of the embankments will be founded upon a thin layer of silty sand overlying a natural deposit of dense till soils encountered after all unsuitable foundation materials have been removed, either in the dry or in the wet. Although no test data is available showing permeability through this deposit, it is locally considered to be relatively impermeable. Since the flow path at the foundation exceeds 140 feet and the heads are small, seepage through the dense in-situ soils will be negligle thus, no special precautions are required.

c. Seepage through Existing Fills. The landward ends of the embankment, on both the Boston and the Charlestown shores, will be constructed upon existing fills which were placed many years ago without regard for placing or compaction techniques. These materials have in the past shown widely variable characteristics with respect to actual in-place permeability. Although the differential heads are small and reversible, and the potential flow paths at maximum head exceed 130 feet, past local experience indicates that minor seepage could occur through these existing materials, several passes of the compaction equipment will be required to densify the upper portions of the in-situ materials. This procedure, together with the filter action of the stone protection courses, will eliminate any adverse effects from minor seepage.

I. STABILITY OF EMBANKMENT, EXCAVATION SLOPES AND EXISTING SLOPES

16. General. Several excavation slopes, an existing slope and an embankment were analyzed for stability against shear failure using circular arc and wedge methods of analyses in accordance with Corps of Engineers Manual EM 1110-2-1902, "Engineering and Design Stability of Earth and Rock Fill Dams". The design shear strengths and unit weights for the organic silt layer were selected on the basis of laboratory test results. The design shear strengths and unit weight for the other embankment and foundation materials were selected on the basis of experience with similar types of materials. Extensive use was made of electronic digital computer in searching for critical circles. The computer solutions for the critical circles were checked by manual computation.

17. Slopes Analyzed.

a. Temporary Excavation Slopes. The existing slope along the Boston shoreline has to be excavated to permit the construction of the temporary bypass channel (See Sections NN and PP on Plate 4-15). Excavation slopes required on the Charlestown shore to allow the dredging of organic silts and other materials were also checked (See Section RR on Plate 4-16). It was assumed that the organic silt was normally consolidated under the existing overburden pressure. These slopes were analyzed for low tide and sudden drawdown conditions. In the latter case it was assumed that the slopes would be still saturated up to high tide Elevation 113 when the low tide occurs at Elevation 100.

b. Embankment. The critical embankment section is presented as Section MM on plate 4-15. The maximum height of embankment which will be constructued on about a 35 foot thick existing sand fill overlying the organic silt,will be less than 8 feet; further away from the river (towards Boston) the embankment tapers to nothing.

(1). Low Tide Analysis. The embankment was analyzed for stability at the end of construction on the assumption that the time required to construct the embankment would be too short to permit the consolidation of the organic silt foundation material under the applied additional embankment load. The conditions of these assumptions were identical to those of the unconsolidated undrained (Q) shear test, the analyses were made using the design shear strengths for the embankment and foundation materials based on this test condition. However, since all the undisturbed organic samples for the unconsolidated undrained (Q) triaxial compression tests were obtained from locations where the existing overburden was limited to a height of about 15 feet, it was judged that higher Q-strengths would exist in the field where the overburden height was of the order of 35 feet. Based on the magnitude of the existing overburden pressure (200-3000+ psf) these Q strengths varied from 300 psf to 1000 psf in the analyses as shown on Plates 4-17 through 4-20. The Q strengths were estimated by using the Mohr strength envelopes shown on Plates B-4 and B-9 in Appendix B.

(2). Rapid Drawdown. Stability analyses were made for sudden drawdown from Elevation 113 to Elevation 100 on the bay side using the composite R-S strength envelope.

c. Existing Slope. The bayside slope on the Boston shore which has been stable ever since it was constructed (more than 100 years ago) was analyzed to check the test data and design assumptions. The section for this slope and its location are shown on Plate 4-15. This section was analyzed for the low tide as well as the rapid drawdown from (Elevation 113 to Elevation 100) conditions. The composite R-S strength envelope was used for the rapid drawdown analysis while the Q-strengths were used for the low tide analysis.

18. Summary of Design Values

a. General. The following is a summary of the various properties of the foundation and embankment materials used in the slope-stability analyses.

b. Unit Weights

Materials	Design Unit Weight in Pounds per Cubic Foot			
	Dry	Moist	Saturated	Submerged
Gravel Fill (Compacted)	--	135	135	71
Gravel Fill (Dumped)	--	135	135	71
Existing Sand Fill	--	110	110	47.6
Organic Silt	--	95	95	32.6
Silty Sand	--	110	110	47.6

c. Design Shear Strengths

Material	S Condition		Q Condition		R Condition	
	Φ Degrees	C T/SF	Φ Degrees	C T/SF	Φ Degrees	C T/SF
Gravel Fills Compacted & Dumped	35	0	35	0	35	0
Existing Sand Fill	33	0	33	0	33	0

Material	S Condition		Q Condition		R Condition	
	ϕ	C	ϕ	C	ϕ	C
	Degrees	T/SF	Degrees	T/SF	Degrees	T/SF
Organic Silt	36 [b]	0	0	0.15 to 0.50 [c] 0.30 [a]	14	0.10
Silty Sand	33	0	33	0	33	0

[a] This lower shear strength was selected for use in the wedge analysis in which the strength of a thin layer of weak material is critical while the average strength was selected for use in the circle analysis where the effect of a thin layer of weak material is less critical.

[b] This value of strength was obtained from the results of consolidated - undrained traixial compression tests with pore pressure measurements. (R-test)

[c] These Q-strengths were estimated from the Mohr strength envelopes for R-tests shown on Plates B-4 and B-9 in Appendix B and are based on the assumption that the organic silt is normally consolidated under the existing overburden pressure (200 to 3000+ psf).

19. Sections Analyzed. The following sections were selected since they combine maximum embankment or slope height with appreciable overburden depth and because the foundation silts occur at these sections:

(a) Excavation Slopes

(1) Slope across temporary bypass channel Boston shore. (Plate 4-15, Section NN)
(2) Slope on Boston shore on basin side. (Plate 4-15, Section PP)
(3) Slope on Charlestown shore after dredging.(Plate 4-16, Section RR)

(b) Embankment

(1) Embankment on Boston shore on bay side. (Plate 4-15, Section MM).

(c) Existing Slope

(1) Slope on Boston shore on bay side (Plate 4-15, Section MM).

20. Results of Stability Analyses. Summaries of the results of the slope and embankment stability analyses and typical analyses are shown on Plate

4-16 through 4-20. The minimum factors of safety against shear failure as determined by the analyses are tabulated below. These minimum factors of safety are considered adequate and the results of the analysis indicate that the slope and embankment are safe against shear failure.

Condition Analyzed	Minimum Factor of Safety
a. Temporary Excavation Slopes	
(1) Bypass Channel Slope (Boston Shore) (Plate 4-18, Section NN)	
(a) Circle Analysis	
(1) Low Tide	1.42
(II) Rapid Drawdown	1.33
(b) Wedge Analysis (Plate 4-20, Section NN)	
(1) Low Tide	1.29
(II) Rapid Drawdown	1.10
(2) Basin side Slope (Boston Shore) (Plate 4-18, Section PP)	
(a) Low Tide	
(1) Circle Analysis	1.52
(b) Rapid Drawdown	
(1) Circle Analysis	1.34
(3) Slope after Dredging (Charlestown Shore) (Plate 4-16, Section RR)	
(a) Low Tide	
(1) Circle Analysis	1.42

b. Embankment

(1) Boston Embankment
(Plate 4-17, Section MM)

(a) Low Tide

(1) Circle 1.33

(b) Rapid Drawdown

(1) Circle Analysis 1.15

c. Existing slope

(1) Bayside Slope (Boston Shore)
(Plate 4-17, Section MM)

(a) Low Tide

(1) Circle Analysis 1.25

(b) Rapid Drawdown

(1) Circle Analysis 1.10

J. EMBANKMENT SETTLEMENT

21. Settlement Estimates

a. General. Except for the organic silt deposit the foundation materials are of very low compressibility. Based on the consolidation characteristics of the organic silt it is estimated that a substantial portion of the settlement due to the embankment load will occur during construction. It is anticipated that the total past construction settlement will not exceed a foot and probably will be less than 6 inches. Settlement measurements will be made during construction and the final paving of the roadway will be delayed until 80% of settlement has occurred.

b. Boston Embankment. Primary consolidation settlements were computed at the various locations along the vehicular viaduct approach embankment and yielded the following results:

Location		Estimated Settlement
Station	Offset	in Inches
53+10	0	4
53+10	33 ft. Left and Right	3 1/2
53+60	0	5
53+60	33 ft. Left and Right	1 1/2

It has been the general local experience that secondary consolidation settlements for such organic soils can sometimes equal 20 percent of the primary settlements.

Percent Primary Consolidation	Time Required in Months
10	0.1
20	0.5
30	1.0
40	1.8
50	2.8
60	4.1
70	5.8
80	8.2
90	12.2
95	14.4

c. Charlestown Embankment. The organic soils will be excavated and replaced with gravel fill. The settlements at this location are anticipated to be negligible.

K. INSTRUMENTATION

22. Settlement plates to monitor settlements during construction will be provided. In view of the relatively low embankment heights, it is considered that the installation of instruments to measure construction pore pressures and slope movements is not required.

PART B - FOUNDATIONS FOR CONCRETE STRUCTURES

A. INTRODUCTION

1. General

a. Description. The major structural portions of the Charles River

navigational traffic; a pumping station to provide control of flood flows, especially during periods of high tide; and a fish ladder for migration of fish through the facility. Gravity sluicing to maintain the basin level during normal conditions will be accomplished through a separate low-level sluicing conduit at low tides. For abnormal conditions, sluicing may also be accomplished through the locks and the lock-filling culverts. A highway viaduct crossing the Charles River over the locks and dam is also part of this project. The locations of these structures are shown on Plate 4-1.

b. Purpose. This portion of the memorandum presents the foundations selected for these structures, and the effects of the subsurface conditions upon their selection. Except for the highway viaduct, the structures can be supported directly on the existing dense till deposit. The determination of the maximum allowable bearing capacity for the concrete structures within the river is dependant upon the practical considerations of the structure and foundation behavior in relation to the foundation soils. The considerations for the foundations of the earth embankments are included in Part A of this memorandum. Design requirements for river earthwork, seepage control under structures, and river bottom protection are presented herein. Also included is a discussion of feasible construction procedures and special requirements necessary to provide uninterrupted flow of navigational traffic and passage of flood flows. Pertinent soil data and the soil conditions at each location are discussed, and criteria for selecting design data such as allowable stresses and sizes are included.

B. SUBSURFACE INVESTIGATIONS

2. Subsurface Explorations. These have been described in Section B.2., Part A; Embankments, of this Memorandum.

3. Laboratory Tests. These have been described in Section B.3, Part A, Embankments, of this Memorandum.

C. CHARACTERISTICS OF FOUNDATION SOILS

4. General Profiles. Generalized soil profiles have been drawn at selected locations to show the existing soil conditions. These are presented on Plate 4-13, and show the embankment limits and the structural locations. These profiles are based on engineering soil reports, which were prepared for all pertinent explorations by the design engineer with the aid of laboratory test data. Included in these reports are descriptions of the soil strata, based on the engineer's examination of the samples and his interpretations of all test results and exploration data. These descriptions include the consistency of the material, estimated or measured percentages of the soil components, color, stratification, presence of foreign matter, geological names and other information of significance in the determination of the characteristics of these materials for design and construction purposes. The results of these soil profiles and sections are as follows:

a. Profile Along the Baseline of the Vehicular Viaduct. The depth of the Charles River along the baseline varies at high tide from a few feet to about 30 feet at the existing navigational channel. The normal low and high tide levels are established at Elevation 100.8 and Elevation 110.2, respectively, based on the M.D.C. Datum. During the monthly tidal cycle, the low and high water levels extend about 2 feet beyond these normal limits. The river bottom deposit consists of very soft sandy to clayey organic silt and organic silty clay with some clamshells hereinafter referred to as organic silt. The thickness of this layer varies from about 5 feet at the Boston shore to approximately 10 feet at the Charlestown shore, and its color ranges from gray-black to black. Underlying the organic silt layer is a continuous stratum of silty sands, varying in thickness from 5 to 10 feet. A continuous deposit of dense to very dense till is next encountered, and its thickness varies between 15 and 25 feet. The bedrock below the till deposit consists of a light gray, soft Argillite with occasional zones of very soft weathered rock. The bedrock deposit has been described in Design Memorandum No. 2, "General Design, Site Geology and Relocations". The portions of the soil profile towards the shorelines under the Boston and Charlestown embankments are described in Part A, "Embankment, "Section C of this Memorandum.

b. Profile Along the Large Boat Lock. The transverse profile shown on Plate 4-13 is drawn along the centerline of the large lock, perpendicular to the baseline of the vehicular viaduct. The river-bottom deposit consists of very soft, dark gray to black sandy organic silts. The thickness of this deposit varies from 5 feet to 10 feet along this profile. The organic silt is underlain by a 5 feet to 30 feet thick layer of silty sand ranging in density from loose to medium dense. Underlying the layer of silty sand is a stratum of dense till. The thickness of this layer ranges between 5 feet and 30 feet, and it is underlain by the Argillite bedrock.

5. Soil Properties.

a. Organic Silt. The properties of this material have been discussed in Section 6.b., Part A of this Memorandum.

b. Silty Sand. The properties of this material have been discussed in Section 6.c., Part A of this Memorandum.

c. Till. The properties of this material have been discussed in Section 6.d., Part A of the Memorandum.

6. Bedrock Properties. The properties of the bedrock have been discussed in Section 7., Part A of this Memorandum.

D. DESIGN CRITERIA

7. Tidal Flood Conditions. The background data and basis for the criteria for design tidal and flood conditions are presented in detail in Design Memorandum No. 1, "Hydrology and Tidal Hydraulics".

8. Settlement. The modulus of vertical subgrade reaction was selected at 340 tons per cubic foot for the 1 foot square bearing plate. After taking the base width of the navigation locks and pumping station structures into

consideration this results in a modulus of vertical subgrade reaction of 85 tcf.

9. Sliding Stability. The coefficient of friction against sliding of the base of structue was adopted at 0.5 with no cohesion.

E. RIVER EARTHWORK

10. Demolition.

a. General. The project site encompasses an area which includes many facilities, some presently useable and others abandoned. On the Boston side these facilities include a boat marina and a portion of the Beverly Street embankment (also called Warren Avenue), now used for vehicular parking and storage. Along the Charlestown shore, the area behind the sea walls is used for transient parking. In addition there is an abandoned pile-supported timber platform which formerly supported a restaurant.

b. Warren Avenue Bridge and Piers. The most conspicuous structure to be demolished within the work area is the abandoned Warren Avenue Bridge. This timber-piled bridge structure, partially destroyed by a fire in the mid-1950's, had previously been used by vehicular and pedestrian traffic between Warren Avenue (and City Square in Charlestown) and Beverly Street (in the North Station area of Boston). Timber piers for navigational control extended downstream to the existing Charlestown Bridge, and upstream beyond the structural piers of the high-level Central Artery Bridge. No effort has since been made to intentionally remove the remains of the bridge and its appurtenant piers. The cobblestone pavement together with utility pipes hung under or against the bridge, have fallen into the river-bottom muds. During periods of high tides and/or storms, pieces of timber have been dislodged and carried into the Inner Harbor as floating debris.

c. Seawalls. Existing seawalls have been used along both the Boston and the Charlestown shores to provide support for filled areas. Portions of the seawalls will be removed, and permanent work subsequently tied into these walls. The seawalls are essentially built of granite blocks laid in mortar, except that one length of wall in Charlestown consists of decayed timber. As-built plans of these seawalls are not available, but local experience has been that these walls consist of granite blocks built upon a timber platform set a foot or two below low water. This platform has been supported in several ways - by timber piles, on in-situ granular soils, on stone ballast or on stone blocks sunk into underlying soft muds. In some cases, the timber platform extends beyond the block wall, and is covered with backfill. Generally, backfill behind the seawall consists of granular materials, although clays, silts and other fills might also be present. At one location along this wall, there has been some loss of materials from behind the seawalls, presumably by loss of ground due to piping, subsidence and/or densification of the backfill or the foundation soils.

11. Excavation in River Area.

a. General. The excavation in the river area will consist of the excavation, removal and disposal of organic silt and of other materials as required for the construction of embankment, channels and approach sections for river flows, and for the concrete river structures. All portions of the abandoned Warren Avenue Bridge and its adjacent piers will be removed prior to starting the river excavation. The limits of river excavation are affected by existing site conditions such as locations of seawalls and bridge piers which are to remain, and by utility systems and other structures crossing the river which must not be interrupted.

b. Excavation of Organic Silt. The organic silt is unsuitable for all construction purposes and will be removed in its entirety for foundation preparation. All excavation operations must insure that adverse amounts of sediments are not placed into suspension in the tidal waters. Removal of the organic silt must be complete where required for stability of the selected cofferdam type. The organic silt existing along the Charlestown shore, together with the overlying fills, will be removed to the extent required for the foundation of the embankment and shoreline development. Excavation or organic silt will also be required along the Boston shore for the temporary by-pass channel, and for areas to receive protection stone to prevent erosion due to wave action and river discharge.

c. Excavation of Granular Materials. Granular materials will be excavated to the extent required for hydraulic considerations, both for the by-pass channel and for the approaches in the upstream and downstream areas. Granular materials and till will be excavated to the extent required for foundation preparation of the river structures.

d. Earthwork within Cofferdammed Areas. The earthwork operations within the cofferdammed areas will consist of the final excavation of the dense till upon which the structures are to be founded or to those grades required for placement of the protection stone. The final 2 feet of excavation for structures will not be made until immediately prior to placing a thin (6 inches) working mat of lean concrete, by which the till will be protected from atmospheric exposure. Where excavations must be carried below the bottom of the foundation to reach the dense till, the below-grade portion will be refilled with lean concrete to the bottom of the structure foundation

e. Effects of Existing Structures. There are several structures within or immediately adjacent to the work area which will affect the earthwork operation. The existing piers for the high-level Central Artery Bridge and the Charlestown Bridge are not to be disturbed. Similarly, utility systems carried across the Charles River upon or adjacent to these structures must not be interrupted. The locations of the new MBTA tunnel and the City of Boston 36-inch ductile steel water conduit, both slightly upstream of the work area, must be permanently and accurately marked to avoid any disturbance.

F. FOUNDATIONS FOR RIVER STRUCTURES

12. Bearing Capacity for Structures. In general, the minimum foundation grades for the structures are below the surface of the dense till. In order to preserve the in-situ integrity of the dense till, a thin working mat of lean concrete (or "mud slab") will be used under all structure foundations. In some locations, over-excavation is required to remove unsuitable materials which overlie the till, and these materials will be replaced by a sub-foundation of lean concrete. All excavations and replacement operations for structures will be done within the dewatered area. Based on local practice, the maximum allowable bearing capacity for the till has been established at 4 tsf. (Boston Building Code allows up to 10 tsf for building structures bearing on till.) Where footings for structures are founded on till, bearing pressures up to 5 or 6 tsf are commonly selected. Based on engineering judgement consistent with local practice, a maximum allowable bearing capacity of 4 tsf for till is considered to be

sufficiently conservative.

13. Boston Marginal Conduit. This conduit, a 90-inch R.C. pipe, will be located adjacent to the Boston bank of the Charles River. The invert grade is indicated at about Elev 92. Since most of the existing soils will have been previously removed for the construction of the locks, this pipe will be placed upon compacted granular fill for the embankment. This conduit has not yet been designed, and it is not known what type of foundations will be required upstream of the project area. This pipe will discharge below the downstream slope stone protection. The details of control structures for gates, final treatment, etc. are covered in Design Memorandum No. 2, "General Design, Site Geology and Relocations.".

G. CHARACTERISTICS OF STRUCTURAL FILLS

14. Gravel Fill and Backfill for Structures.

a. General. Materials to be used for fill and backfill beneath and adjacent to structures will consist of well-graded sandy gravel or gravelly sand.

b. Gradation. Gravel shall be well-graded throughout the entire range of particle sizes, and shall be graded so as to meet the following requirements:

U.S. Standard Sieve Designation	Percent Passing, by Weight
3 inch	100
2 inch	90 - 100
1 inch	75 - 90
No. 4	40 - 70
No. 40	20 - 50
No. 200	0 - 8

c. Design Values. The following properties were adopted for compacted gravel in the design of concrete structures:

Angle of internal friction = 32 degrees

Moist Unit Weight	=	125 pcf
Saturated Unit Weight	=	125 pcf

H. CONSIDERATIONS OF RIVER CONSTRUCTION

15. General. The foundation preparation of all river structures will be in the dry within cofferdammed areas. The sequence of construction and the locations of outboard faces of cofferdams will have to be limited so that the conditions of navigational traffic and river flow are maintained. The final excavation lift to or within till will be immediately followed by the placement of the thin working mat of lean concrete. Construction of the river structures will then proceed within the cofferdams. When all work below the high tide level is completed, the cofferdam can then be flooded and breached.

16. Sequence of Construction.

a. River Excavation. Excavation of the organic silt and granular materials may be accomplished either in the wet or in the dry. Prior to the completion of final foundation preparation, cofferdams will be installed and dewatered.

b. Stage I. Because of the configuration of the structures and the need to continuously accommodate river flows and navigational traffic, 2 stages of construction will be required as shown on Plate 4-23. Structures within the first stage include the fish ladder and sluiceway, the pumping station and the large boat lock, and a portion of the small boat lock structure. Also included in Stage I is that section of embankment extending toward Charlestown. The Stage I cofferdam will extend into the Charlestown shoreline and the by-pass channel for navigation and flows will be excavated and maintained along the Boston shoreline. Access into the cofferdammed area will be by earth ramps from the Charlestown side. There is sufficient space on the Charlestown shore beneath the Central Artery vehicular viaduct for the contractor's work area and storage areas. When the structures and the Charlestown embankment are sufficiently complete and the pumping station and large lock waterways are open, the Stage I cofferdam will be flooded and removed.

c. Stage II. Upon removal of the Stage I cofferdam, navigational traffic and the tidal and river flows will be diverted through the completed structures. The Stage II cofferdam will then be installed and dewatered, and the remainder of the small boat locks will be constructed. A portion of the Boston embankment, including the discharge end of the Boston Marginal Conduit, will also be built within the Stage II.

17. Cofferdam Schemes.

a. General. A review was made of the several cofferdam schemes which might be utilized in order to determine which scheme might be more economical and/or feasible. It was concluded that, while several schemes could be possibly be employed, a cellular cofferdam would be most feasible. The design of the Cofferdam will be covered in Design Memorandum No. 8 "Cofferdams".

b. Schematic Preliminary Layout. A schematic preliminary layout of a workable cellular cofferdam scheme is shown on Plate 4-23. The final layout will be in Design Memorandum No. 8, "Cofferdams". Earth-filled cells would enclose the required work areas for Stages I and II. This arrangement would provide access from the tops of the cells into both work areas during the respective stages, as well as sufficient space for dewatered foundation preparation. It would also allow the required passage of traffic and river flows around the Stage I limits within a temporary by-pass channel. All organic soil will be removed from the cofferdam cells before they are filled with gravel.

I. SEEPAGE CONTROL

18. River Structures. The final preparation for the foundations of the concrete structures will be made within a cofferdammed area. The foundations will bear directly upon the relatively impervious till, which will be protected by a thin mat of lean concrete. Because of these operations and the small and reversible differential heads, seepage protection beneath the structures is not required.

J. RIVER-BOTTOM PROTECTION

19. General. Scour protection generally consists of a 2 foot thick layer of 5 to 300 lbs (Type I) stone dumped on a 1 foot filter layer of gravel where

placement will be on dry areas and on a 3 foot layer where placement will be on submerged areas. The protection will extend 40 feet on both the upstream and downstream side of river structures except downstream of the large lock where it will extend 260 feet. Downstream of the large lock the stone size will be graded from 50 to 500 pounds (Type II). Type I stone protection will be provided around the existing piers of John Fitzgerald Expressway bridge and the Charlestown bridge. This protection will extend 8 feet beyond the perimeter of the pier footings (or pile caps). Steel sheeting will be driven on 3 sides of the John Fitzgerald Expressway piers located near the temporary by pass channel. The temporary by pass channel will be protected against scour with a 2 feet thick layer of 50 to 500 lbs. (Type II) dumped stone.

K. FOUNDATIONS FOR VEHICULAR VIADUCT

20. General. The foundations for the vehicular viaduct are included in Design Memorandum No. 6, "Vehicular Viaduct". Portions of the viaduct which are not resting on the concrete structures in the river will be supported on end-bearing steel H-piles.

L. FOUNDATIONS FOR FENDER PIERS AND TRAINING STRUCTURES

21. General. A discussion of the foundation for the fender piers and training structures is presented in Design Memorandum No. 7, "Navigation Locks and Facilities". These consist mostly of treated timber piles and some steel H-piles.

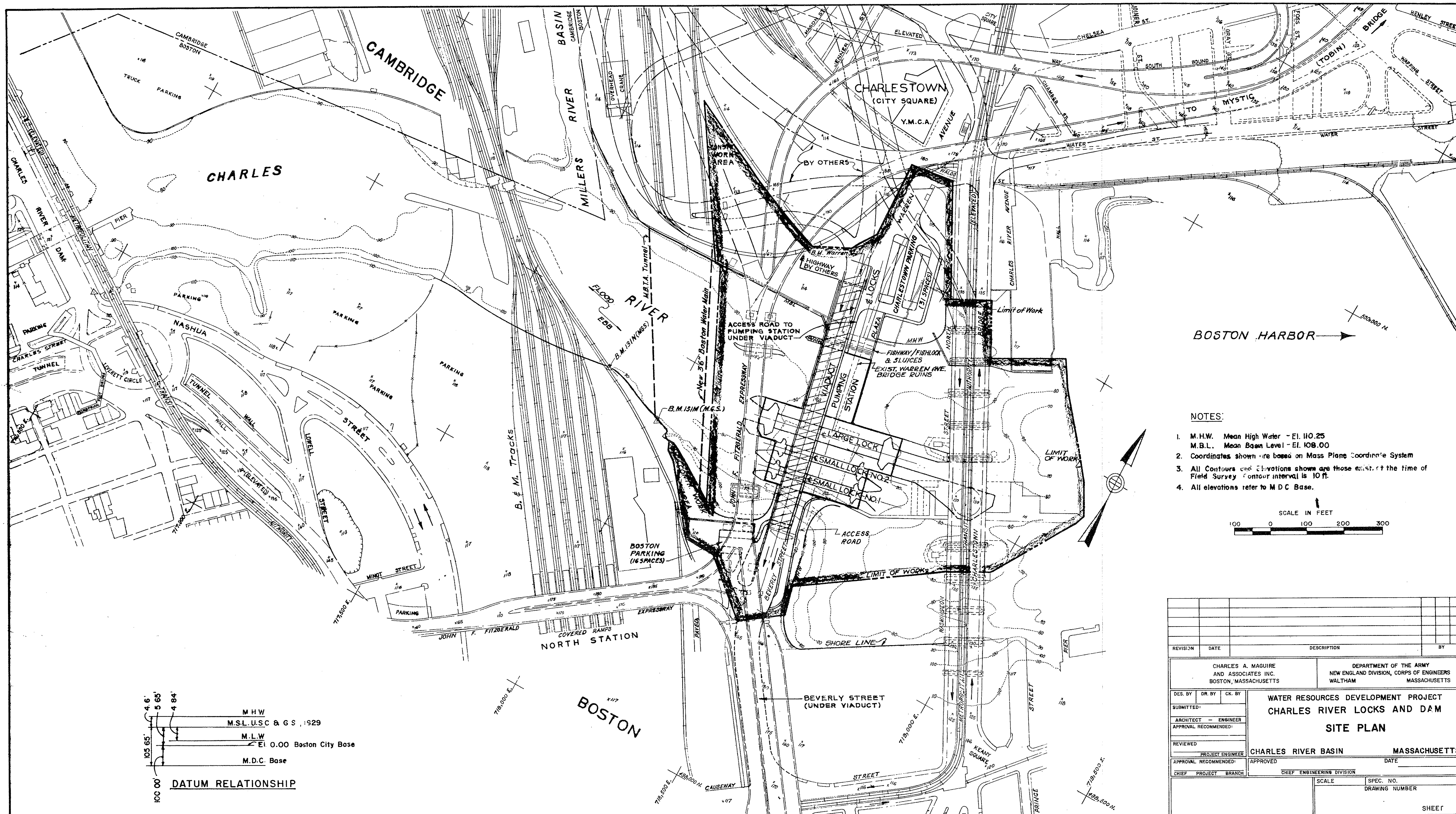
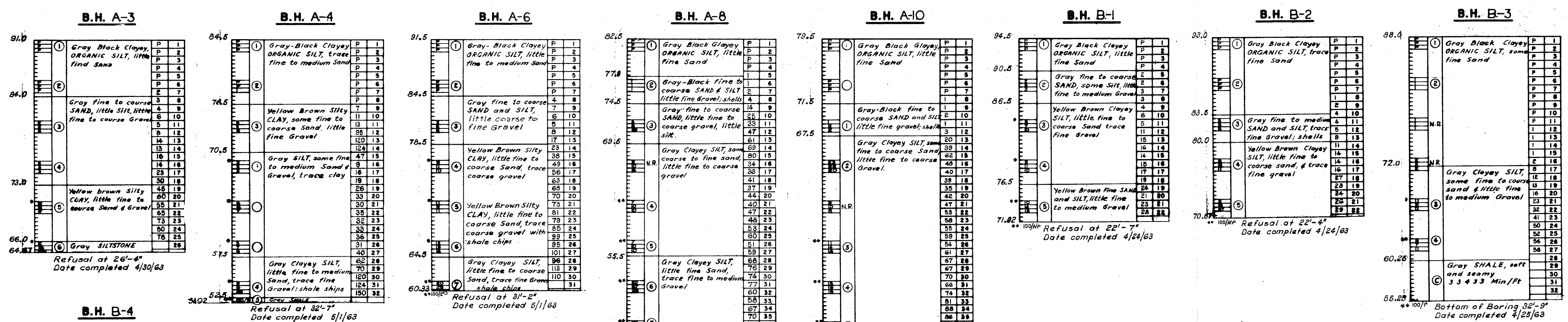
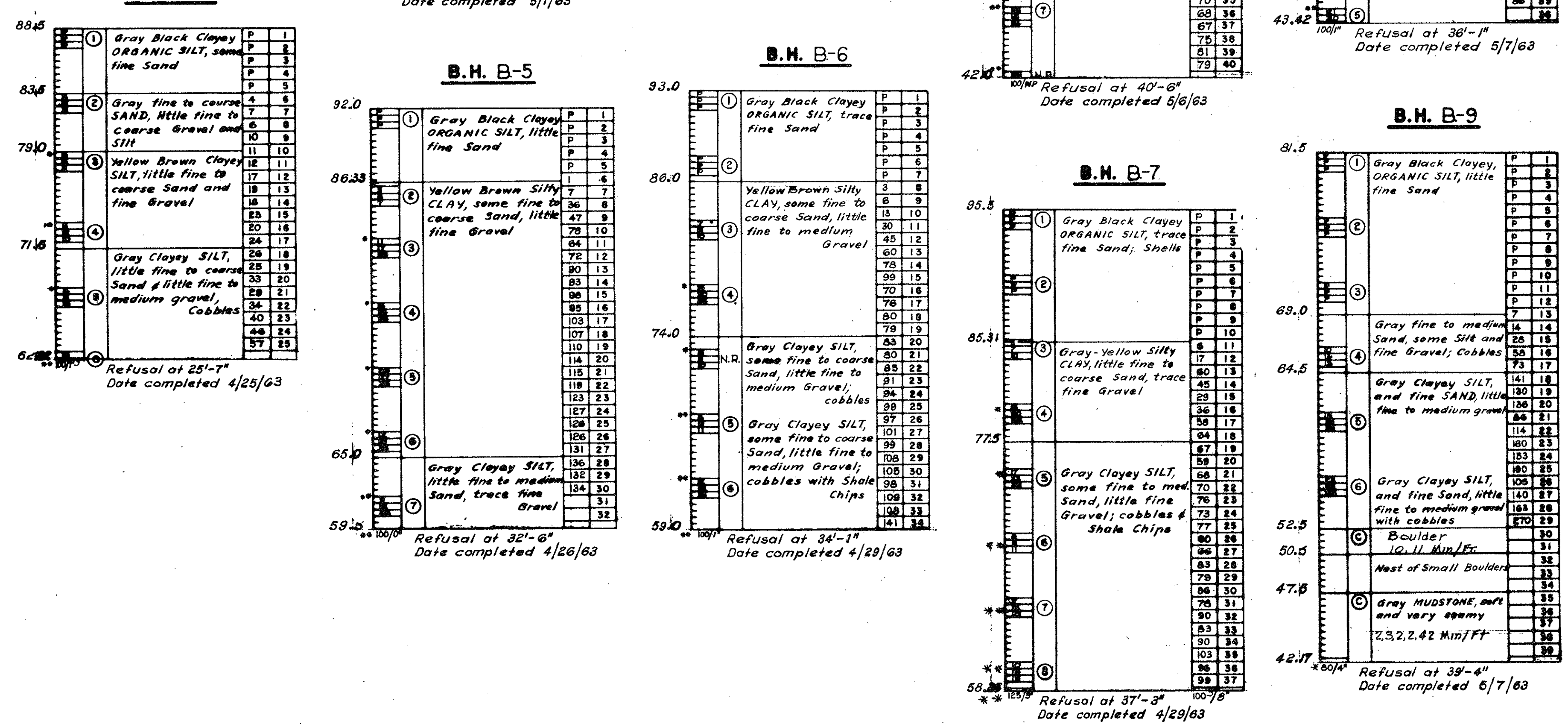

CAMBRIDGE
CHARLES
CHARLESTOWN (CITY SQUARE)
Y.M.C.A.
BOSTON HARBOR
BOSTON
NORTH STATION
MILLERS RIVER
RIVER
FLOOD
EBB
B. & M. Tracks
New 36" Boston Water Main
M.B.T.A. Tunnel
ACCESS ROAD TO PUMPING STATION UNDER VIADUCT
HIGHWAY BY OTHERS
BY OTHERS
CHARLESTOWN PARKING (31 SPACES)
BOSTON PARKING (16 SPACES)
FISHWAY/FISHLOCK & SLUICES
EXIST. WARREN AVE. BRIDGE RUINS
PUMPING STATION
VIADUCT
LARGE LOCK
SMALL LOCK NO. 2
SMALL LOCK NO. 1
ACCESS ROAD
LIMIT OF WORK
Limit of Work
SHORE LINE
BEVERLY STREET (UNDER VIADUCT)
JOHN F. FITZGERALD EXPRESSWAY
NASHUA STREET
LOWELL STREET
MINOT STREET
CAUSEWAY STREET
WATER ST.
CHELSEA ST.
HENLEY STREET
WARREN AVE.
CHARLES RIVER AVENUE
TO MYSTIC (TOBIN) BRIDGE
NOTES:
1. M.H.W. Mean High Water - El. 110.25
M.B.L. Mean Basin Level - El. 108.00
2. Coordinates shown are based on Mass Plane Coordinate System
3. All Contours and Elevations shown are those exist. at the time of Field Survey Contour interval is 10 ft.
4. All elevations refer to M D C Base.
SCALE IN FEET
100 0 100 200 300
M H W
M.S.L. U.S.C. & G.S. 1929
M.L.W
El 0.00 Boston City Base
M.D.C. Base
4.6'
5.65'
4.84'
105.65'
100.00'
DATUM RELATIONSHIP
REVISION DATE DESCRIPTION BY
CHARLES A. MAGUIRE AND ASSOCIATES INC. BOSTON, MASSACHUSETTS
DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION, CORPS OF ENGINEERS WALTHAM MASSACHUSETTS
WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
SITE PLAN
CHARLES RIVER BASIN MASSACHUSETTS
DES. BY DR. BY CK. BY
SUBMITTED
ARCHITECT - ENGINEER
APPROVAL RECOMMENDED
REVIEWED
PROJECT ENGINEER
APPROVAL RECOMMENDED
CHIEF PROJECT BRANCH
APPROVED DATE
CHIEF ENGINEERING DIVISION
SCALE SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 4-1

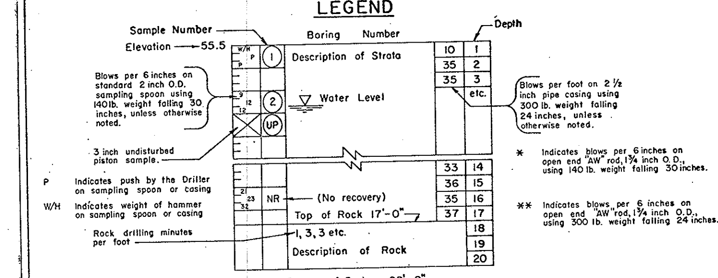

CHARLESTOWN
BOSTON
MATCH LINE
CHARLESTOWN BRIDGE
CORRUGATED ASBESTOS WAREHOUSE
Charlestown Bridge (Ramp)
METAL QUONSET HUT (Removed)
LOBSTER HOUSE (Removed) 1½ Story Wood
(Removed) 2½ Story Wood
PUMPING STATION
Fishway, Flashboards and Sluiceway
Vehicular Viaduct
WARREN AVENUE
BEVERLY ST.
℄ Large Lock
℄ Small Lock No.1
℄ Small Lock No.2
℄ Locks
Face of Fender
Wood Pier
RIPRAP
Retaining Wall
Cobble stone rip-rap meets overhead planking
Top of Planking El. 114.3
Top of Wall Elev. 114.3±
Ground Elev. 113.8±
Abutment El. 112.6
Top Elev. 94.80
Top Elev. 112.70
E 718,000
E 718,800
N 499,400
N 499,000
SCALE IN FEET
1" = 40'
LEGEND
A-3
Locations of borings made in 1963 by American Drilling & Boring Co.,Inc., East Providence R.I. for Charles A. Maguire & Associates, Inc. Boston, Mass.
FD-3
Locations of borings made in 1970 by Corps of Engineers, N.E. Division.
NOTES:
1. Boring Samples may be seen at the offices of the U.S. Army Corps of Engineers, Waltham, Mass.
2. Elevations in plan refer to Metropolitan District Commission Base.
DATUM RELATIONSHIP
M.S.L. U.S.C. & G.S.
105.65'
M.D.C. BASE
REVISION
DATE
DESCRIPTION
BY
CHARLES A. MAGUIRE AND ASSOCIATES INC. WALTHAM, MASSACHUSETTS
DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION, CORPS OF ENGINEERS WALTHAM MASSACHUSETTS
DES. BY
DR. BY
CK. BY
SUBMITTED:
ARCHITECT - ENGINEER
APPROVAL RECOMMENDED:
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF PROJECT BRANCH
WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
PLAN OF FOUNDATION EXPLORATIONS
CHARLES RIVER BASIN
MASSACHUSETTS
APPROVED
DATE
CHIEF ENGINEERING DIVISION
SCALE: 1" = 40'
SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 4-2





LEGEND



NOTES

1. Ground water levels indicated on the drawings are those existing at the time subsurface investigations were made and do not necessarily represent permanent ground water levels. It shall be the Contractors responsibility to determine annual and seasonal variations in ground water levels which may effect his work. Variations in existing ground water levels from those indicated on the drawings shall under no conditions constitute grounds for revision in contract price or completion date.

2. Borings, probings, soundings, etc. were taken for the purpose of design and indicate subsurface conditions only at the locations shown. The Contractor shall be aware that subsurface conditions encountered during construction may vary from those shown on the plans. See specifications.

3. Descriptions of estimated amount of materials indicated on the borings refer to percentage by weight as follows: Trace 0–10 %, Little 10–20 %, Some 20–35 %.

NOTE:
For additional notes and locations of borings see Plate 4-2

GRAPHIC SCALE

1" = 5'   5'   0   5'   10'

REVISION	DATE	DESCRIPTION	BY

CHARLES A. MAGUIRE AND ASSOCIATES INC.
WALTHAM, MASSACHUSETTS

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM MASSACHUSETTS

DES. BY | DR. BY | CK. BY

SUBMITTED:

ARCHITECT – ENGINEER

APPROVAL RECOMMENDED:

REVIEWED:

PROJECT ENGINEER

APPROVAL RECOMMENDED:

CHIEF PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
RECORD OF SUBSURFACE EXPLORATIONS NO. 1

CHARLES RIVER BASIN MASSACHUSETTS

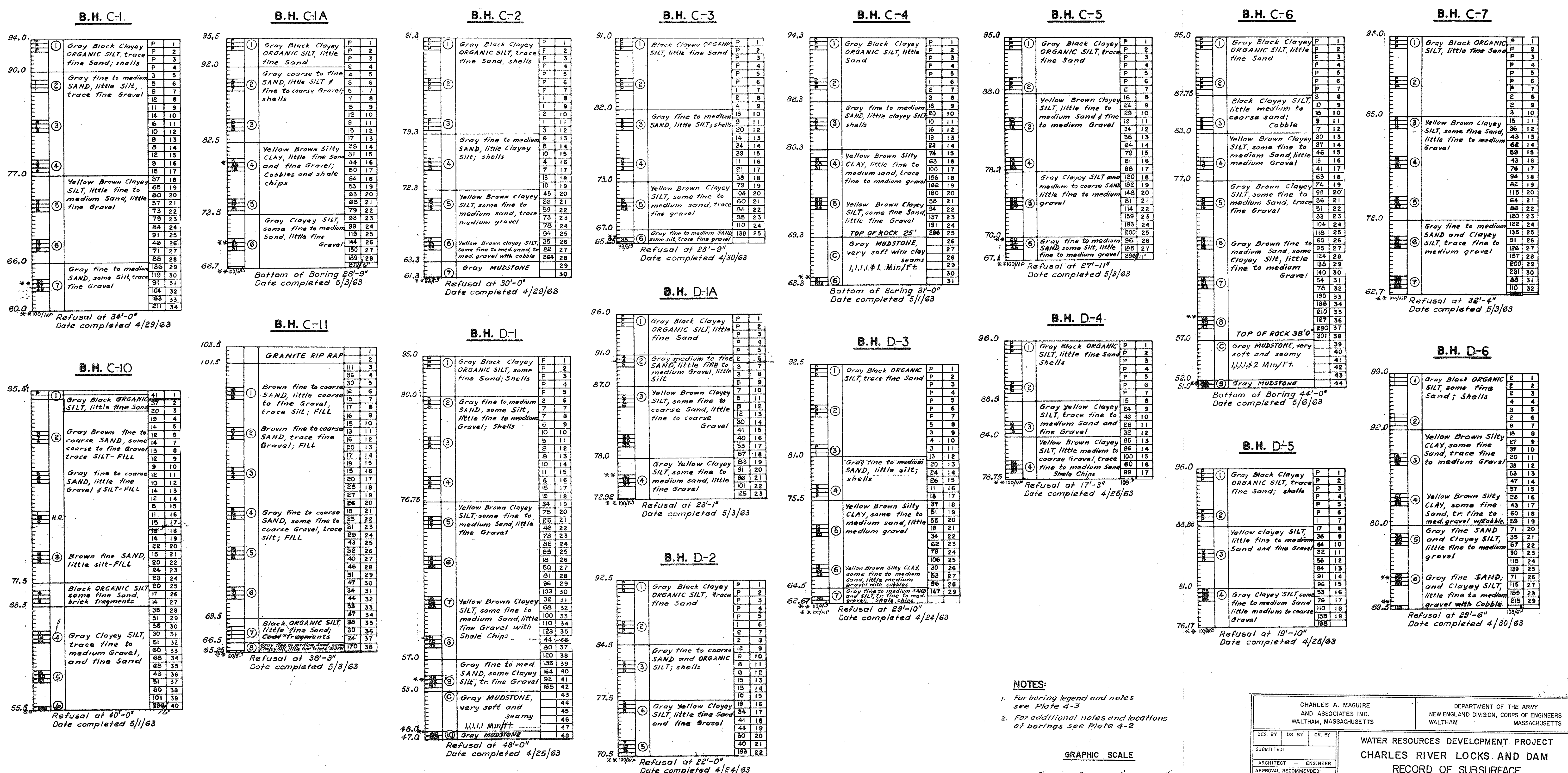
APPROVED DATE

CHIEF ENGINEERING DIVISION

SCALE | SPEC. NO.

DRAWING NUMBER

SHEET



NOTES:

1. For boring legend and notes see Plate 4-3
2. For additional notes and locations of borings see Plate 4-2

GRAPHIC SCALE

1" = 5' 5' 0 5' 10'

REVISION	DATE	DESCRIPTION	BY

CHARLES A. MAGUIRE AND ASSOCIATES INC. WALTHAM, MASSACHUSETTS

DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION, CORPS OF ENGINEERS WALTHAM MASSACHUSETTS

DES. BY | DR. BY | CK. BY

SUBMITTED: ARCHITECT — ENGINEER

APPROVAL RECOMMENDED:

REVIEWED: PROJECT ENGINEER

APPROVAL RECOMMENDED: CHIEF PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
RECORD OF SUBSURFACE EXPLORATIONS NO. 2

CHARLES RIVER BASIN MASSACHUSETTS

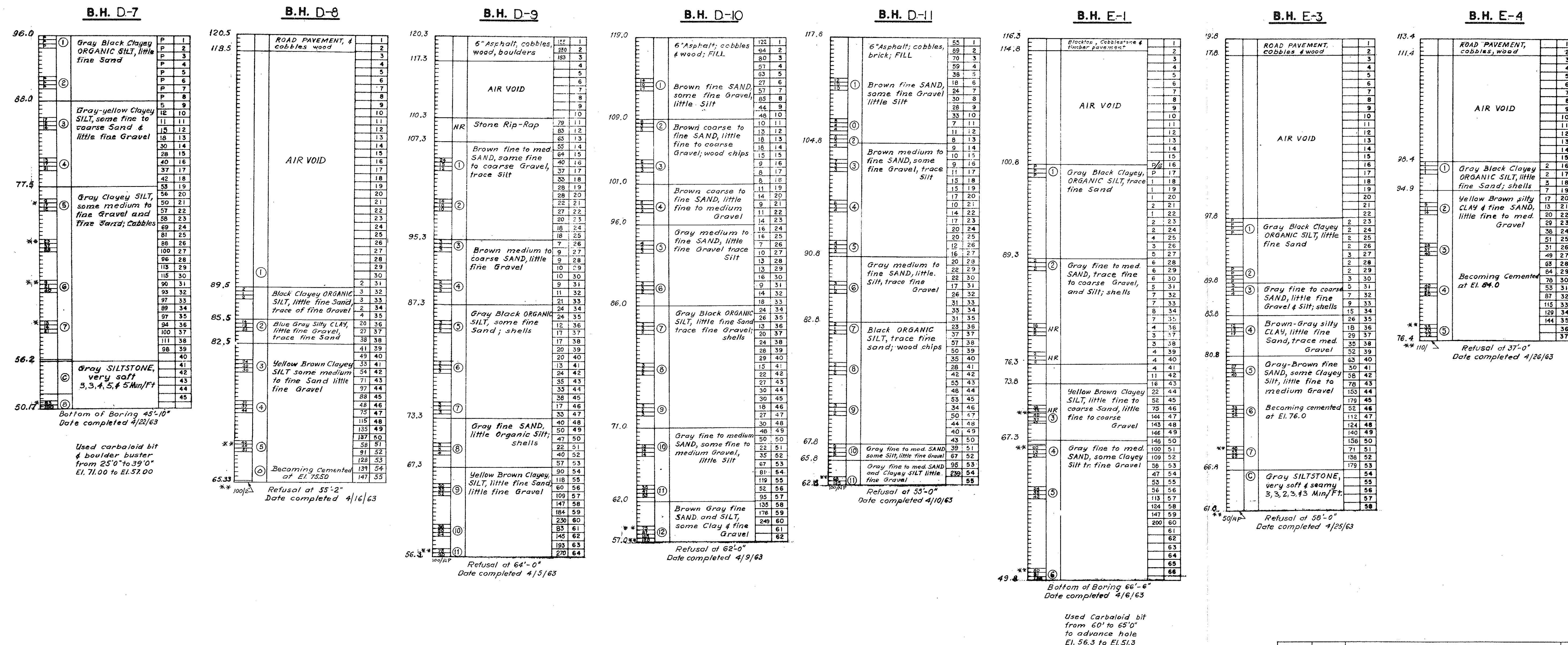
APPROVED DATE

CHIEF ENGINEERING DIVISION

SCALE | SPEC. NO. | DRAWING NUMBER

SHEET

PLATE 4-4



NOTES:

1. For boring legend and notes see Plate 4-3
2. For additional notes and locations of borings see Plate 4-2

GRAPHIC SCALE

1" = 5' 5' 0 5' 10'

REVISION	DATE	DESCRIPTION	BY

CHARLES A. MAGUIRE AND ASSOCIATES INC. WALTHAM, MASSACHUSETTS

DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION, CORPS OF ENGINEERS WALTHAM MASSACHUSETTS

DES. BY | DR. BY | CK. BY

SUBMITTED:

ARCHITECT - ENGINEER

APPROVAL RECOMMENDED:

REVIEWED:

PROJECT ENGINEER

APPROVAL RECOMMENDED:

CHIEF PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT

CHARLES RIVER LOCKS AND DAM

RECORD OF SUBSURFACE EXPLORATIONS NO. 3

CHARLES RIVER BASIN MASSACHUSETTS

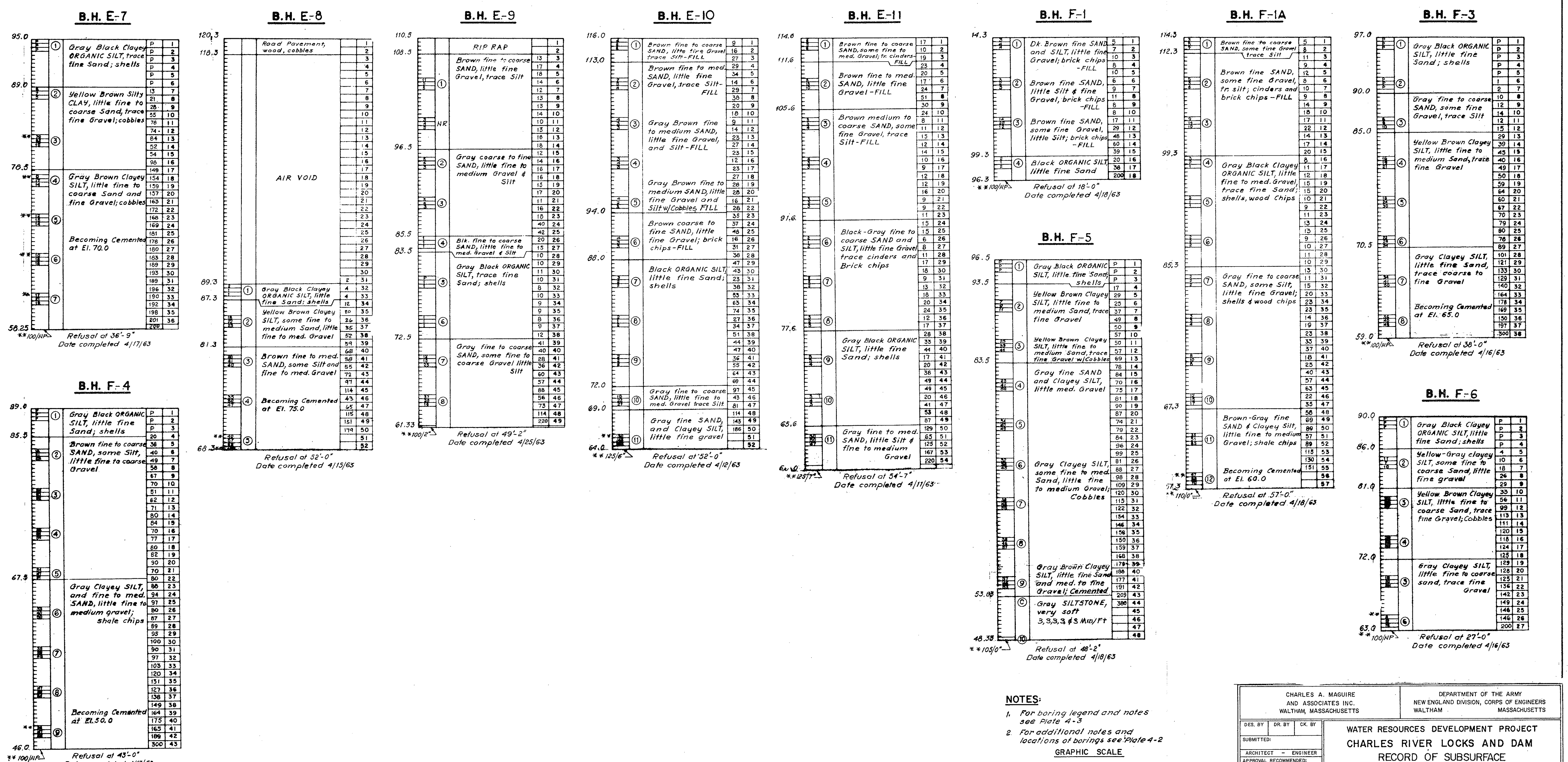
APPROVED DATE

CHIEF ENGINEERING DIVISION

SCALE | SPEC. NO.

DRAWING NUMBER

SHEET



NOTES:

1. For boring legend and notes see Plate 4-3
2. For additional notes and locations of borings see Plate 4-2

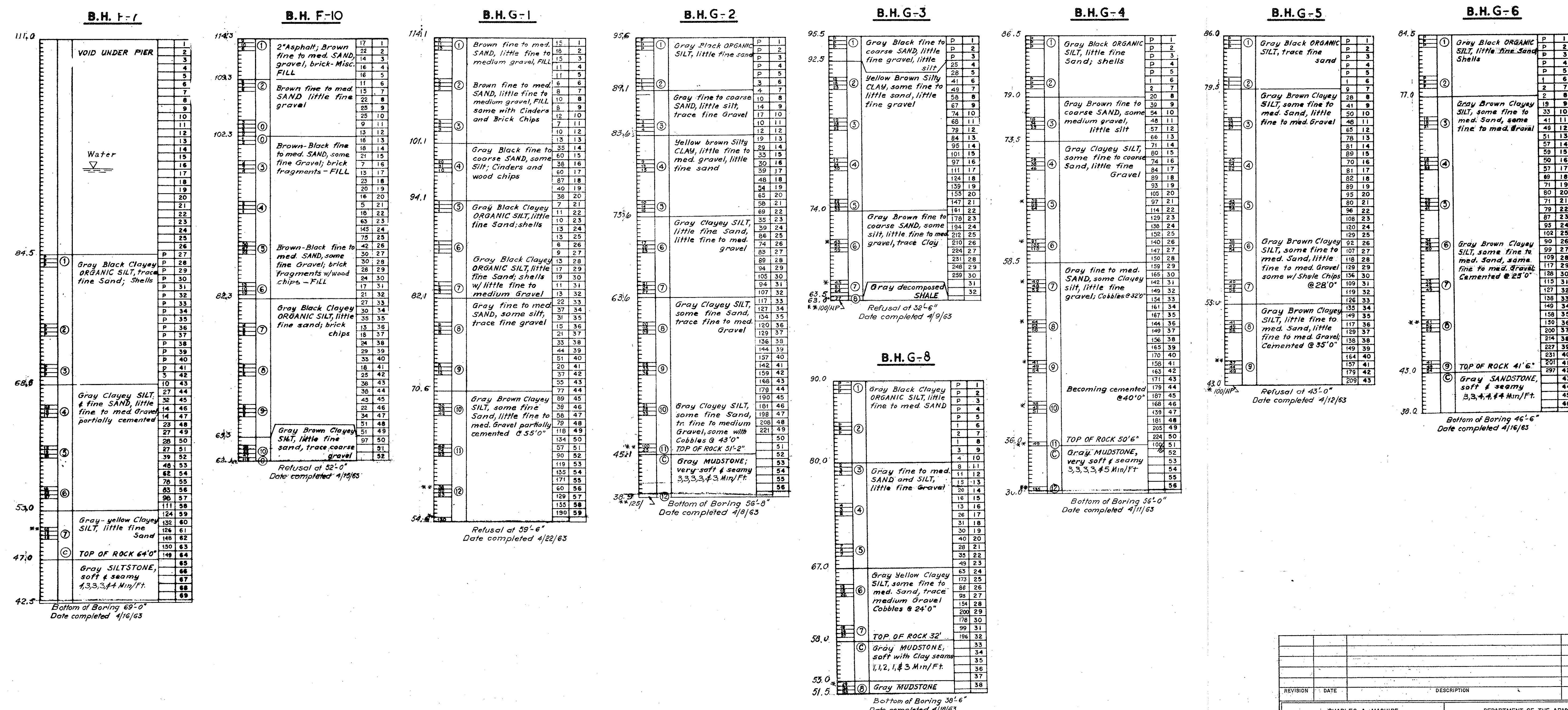
GRAPHIC SCALE

1" = 5' 5' 0 5' 10'

CHARLES A. MAGUIRE AND ASSOCIATES INC. WALTHAM, MASSACHUSETTS	DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION, CORPS OF ENGINEERS WALTHAM MASSACHUSETTS
DES. BY / DR. BY / CK. BY	WATER RESOURCES DEVELOPMENT PROJECT
SUBMITTED: ARCHITECT - ENGINEER	CHARLES RIVER LOCKS AND DAM
APPROVAL RECOMMENDED:	RECORD OF SUBSURFACE EXPLORATIONS NO. 4
REVIEWED: PROJECT ENGINEER	CHARLES RIVER BASIN MASSACHUSETTS
APPROVAL RECOMMENDED: CHIEF PROJECT BRANCH	APPROVED DATE CHIEF ENGINEERING DIVISION
	SCALE / SPEC. NO. / DRAWING NUMBER / SHEET

REVISION	DATE	DESCRIPTION	BY

PLATE 4-6



NOTES:

1. For boring legend and notes see Plate 4-3
2. For additional notes and locations of borings see Plate 4-2

REVISION	DATE	DESCRIPTION	BY

CHARLES A. MAGUIRE AND ASSOCIATES INC. WALTHAM, MASSACHUSETTS

DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION, CORPS OF ENGINEERS WALTHAM MASSACHUSETTS

DES. BY | DR. BY | CK. BY

SUBMITTED:

ARCHITECT - ENGINEER

APPROVAL RECOMMENDED:

REVIEWED:

PROJECT ENGINEER

APPROVAL RECOMMENDED:

CHIEF PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
RECORD OF SUBSURFACE
EXPLORATIONS NO. 5

CHARLES RIVER BASIN MASSACHUSETTS

APPROVED DATE

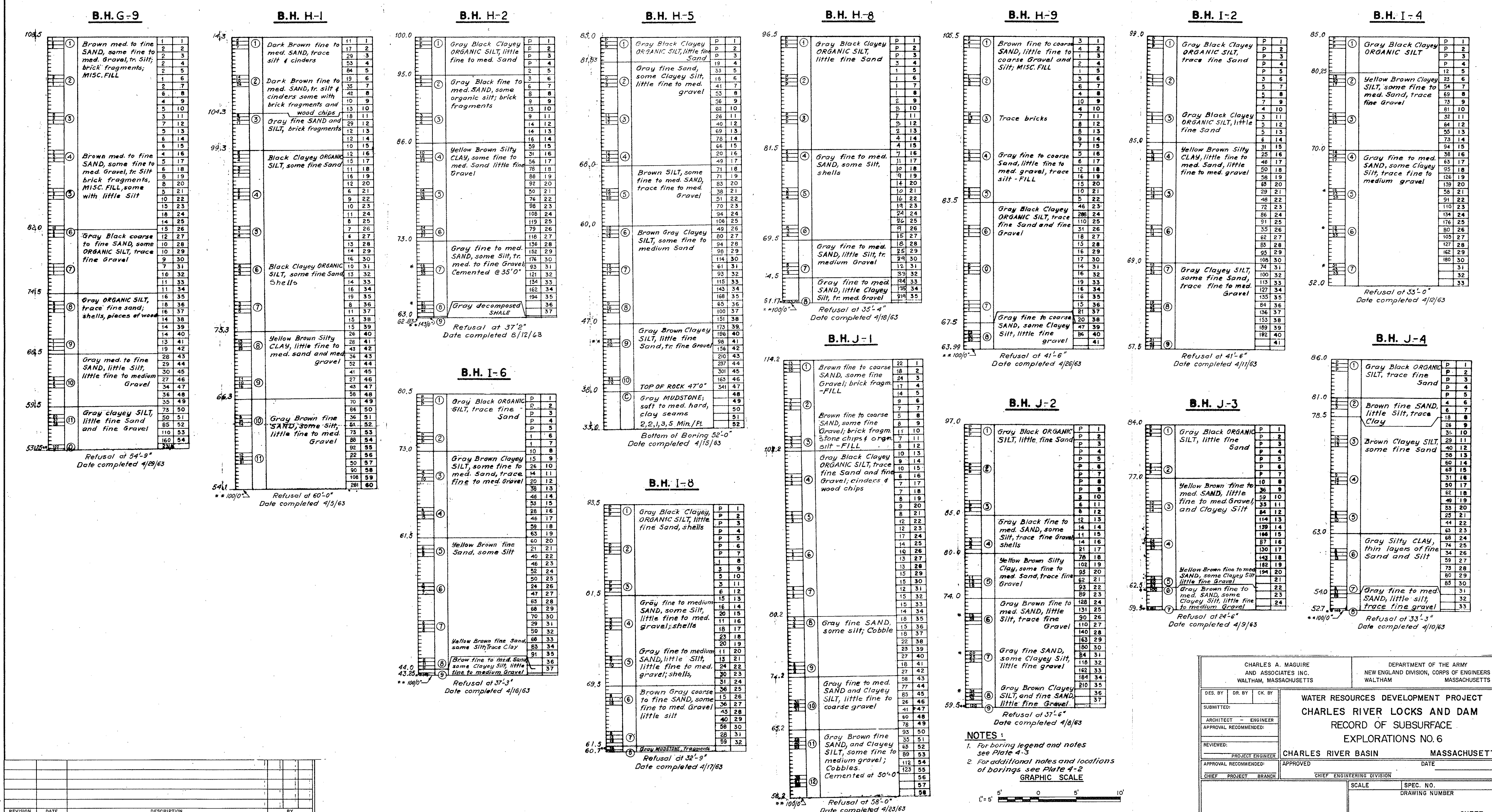
CHIEF ENGINEERING DIVISION

SCALE | SPEC. NO.

DRAWING NUMBER

SHEET

PLATE 4-7



NOTES:
1. For boring legend and notes see Plate 4-3
2. For additional notes and locations of borings see Plate 4-2

GRAPHIC SCALE
1" = 5' 5' 0 5' 10'

REVISION	DATE	DESCRIPTION	BY

CHARLES A. MAGUIRE AND ASSOCIATES INC. WALTHAM, MASSACHUSETTS

DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION, CORPS OF ENGINEERS WALTHAM MASSACHUSETTS

DES. BY | DR. BY | CK. BY

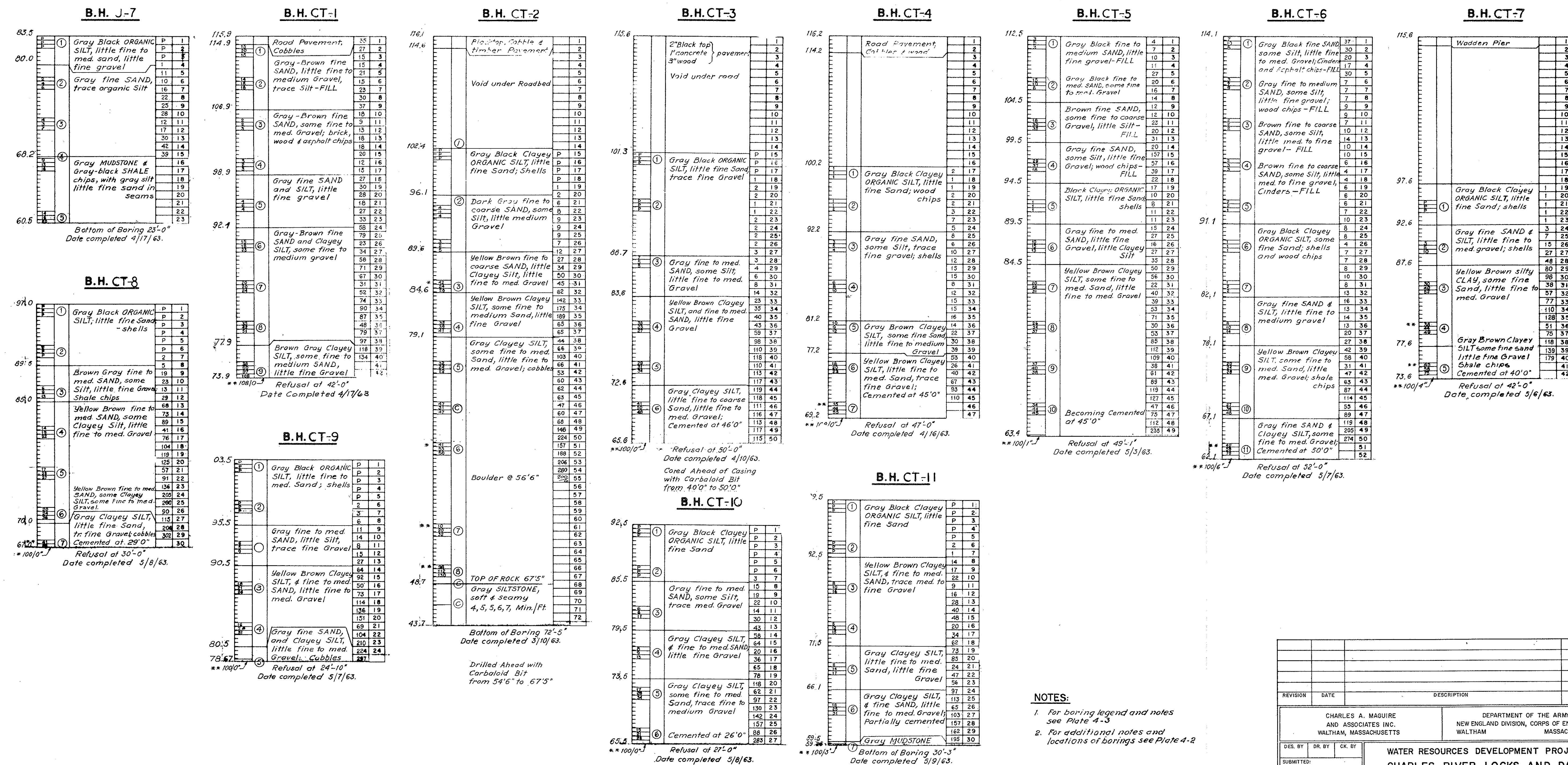
SUBMITTED:
ARCHITECT - ENGINEER
APPROVAL RECOMMENDED:
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
RECORD OF SUBSURFACE EXPLORATIONS NO. 6
CHARLES RIVER BASIN MASSACHUSETTS

APPROVED | DATE
CHIEF ENGINEERING DIVISION

SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 4-8



NOTES:

1. For boring legend and notes see Plate 4-3
2. For additional notes and locations of borings see Plate 4-2

GRAPHIC SCALE

1" = 5'

REVISION	DATE	DESCRIPTION	BY

CHARLES A. MAGUIRE AND ASSOCIATES INC. WALTHAM, MASSACHUSETTS

DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION, CORPS OF ENGINEERS WALTHAM MASSACHUSETTS

DES. BY | DR. BY | CK. BY

SUBMITTED:

ARCHITECT - ENGINEER

APPROVAL RECOMMENDED:

REVIEWED:

PROJECT ENGINEER

APPROVAL RECOMMENDED:

CHIEF PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT

CHARLES RIVER LOCKS AND DAM

RECORD OF SUBSURFACE EXPLORATIONS NO. 7

CHARLES RIVER BASIN MASSACHUSETTS

APPROVED DATE

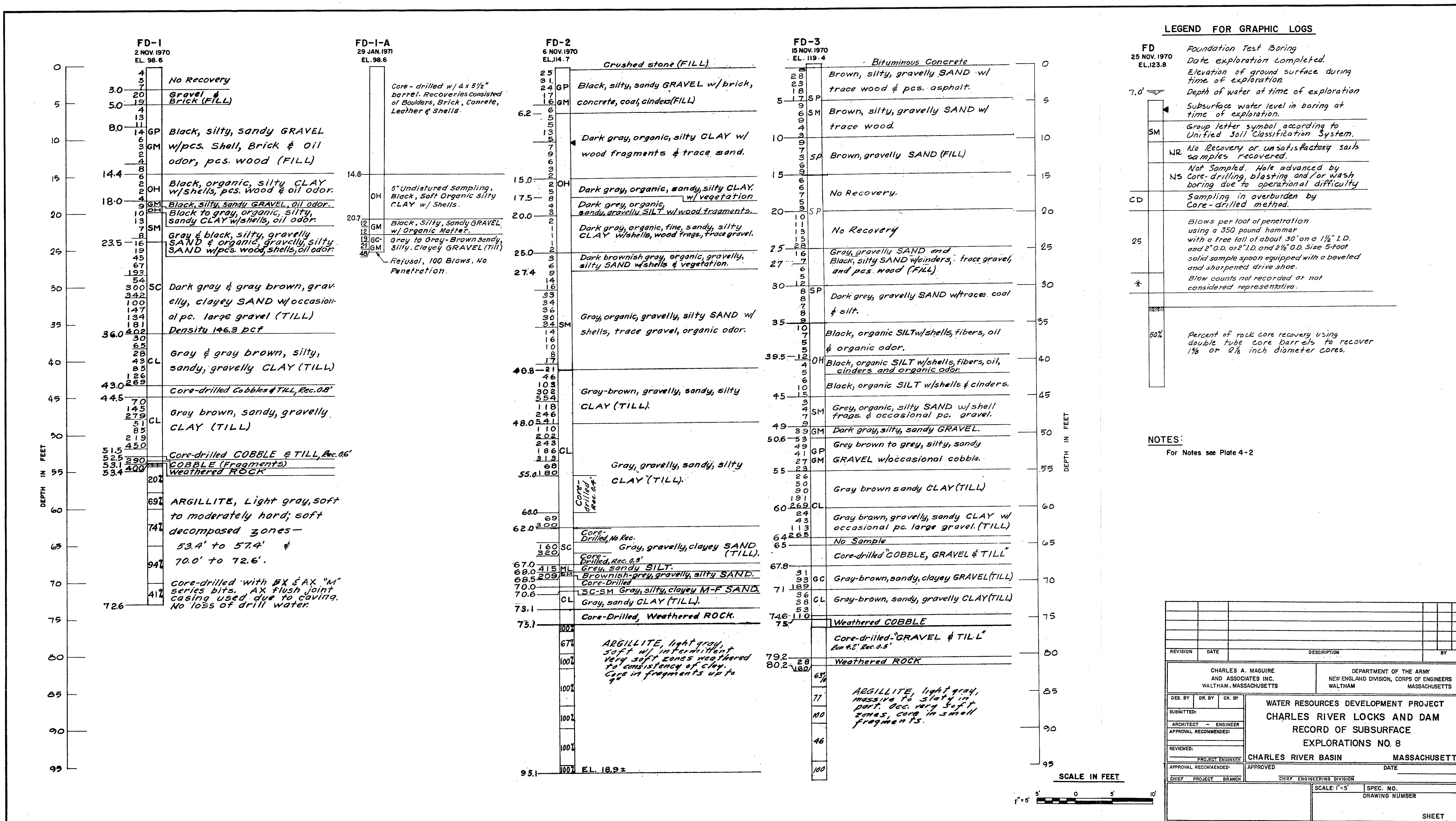
CHIEF ENGINEERING DIVISION

SCALE | SPEC. NO.

DRAWING NUMBER

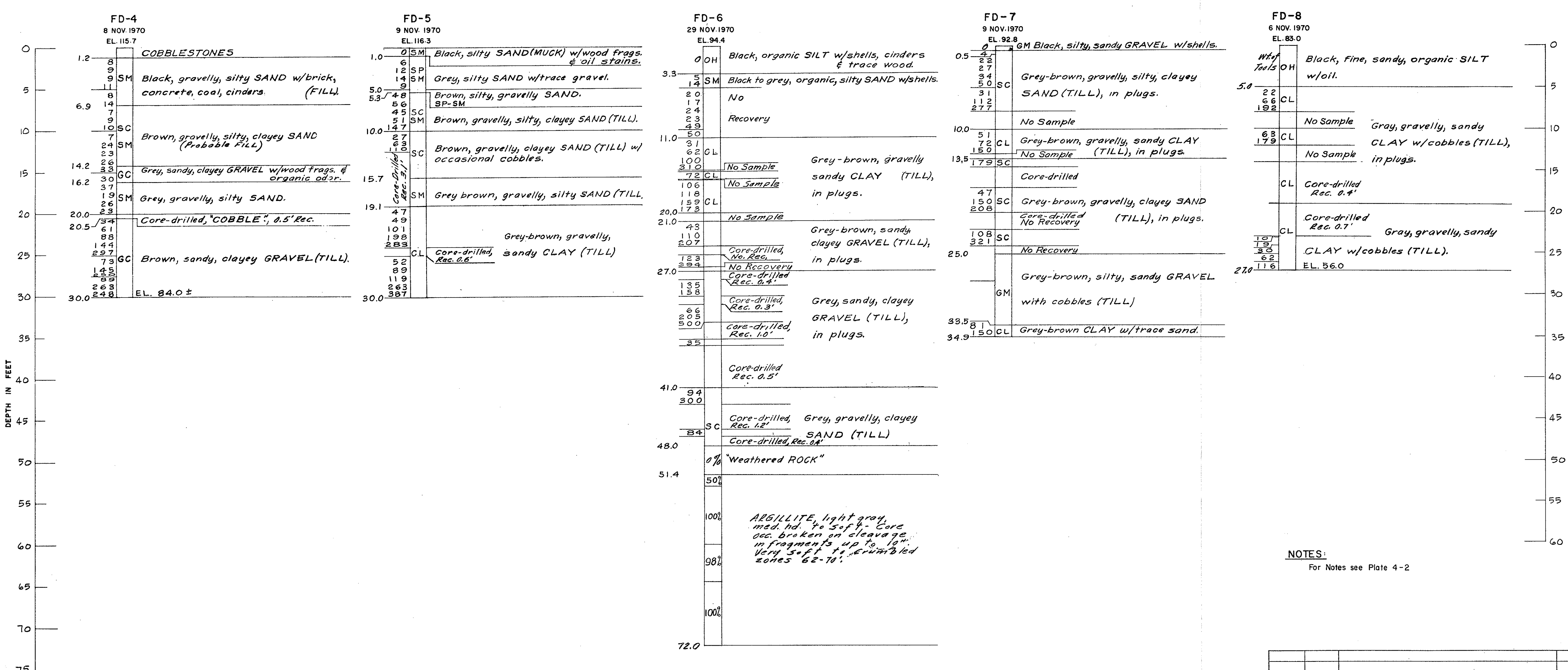
SHEET

PLATE 4-9



FD-1
2 NOV. 1970
EL. 98.6
No Recovery
Gravel & Brick (FILL)
Black, silty, sandy GRAVEL w/pcs. Shell, Brick & Oil odor, pcs. wood (FILL)
Black, organic, silty CLAY w/shells, pcs. wood & oil odor.
Black, silty, sandy GRAVEL, oil odor.
Black to gray, organic, silty, sandy CLAY w/shells, oil odor.
Gray & black, silty, gravelly SAND & organic, gravelly, silty SAND w/pcs. wood, shells, oil odor.
Dark gray & gray brown, gravelly, clayey SAND w/occasional pc. large gravel (TILL)
Density 146.3 pcf
Gray & gray brown, silty, sandy, gravelly CLAY (TILL)
Core-drilled Cobbles & TILL, Rec. 0.8'
Gray brown, sandy, gravelly CLAY (TILL)
Core-drilled COBBLE & TILL, Rec. 0.6'
COBBLE (Fragments)
Weathered ROCK
ARGILLITE, Light gray, soft to moderately hard; soft decomposed zones— 53.4' to 57.4' & 70.0' to 72.6'.
Core-drilled with BX & AX "M" series bits. AX flush joint casing used due to caving. No loss of drill water.
FD-1-A
29 JAN. 1971
EL. 98.6
Core-drilled w/ 4 x 5½" barrel. Recoveries consisted of Boulders, Brick, Concrete, Leather & Shells.
5" Undisturbed Sampling, Black, Soft Organic Silty CLAY w/ Shells.
Black, Silty, Sandy GRAVEL w/ Organic Matter.
Gray to Grey-Brown Sandy, Silty, Clayey GRAVEL (Till)
Refusal, 100 Blows, No Penetration.
FD-2
6 NOV. 1970
EL. 114.7
Crushed stone (FILL)
Black, silty, sandy GRAVEL w/brick, concrete, coal, cinders (FILL)
Dark gray, organic, silty CLAY w/ wood fragments & trace sand.
Dark gray, organic, sandy, silty CLAY w/ vegetation
Dark grey, organic, sandy, gravelly SILT w/wood fragments.
Dark gray, organic, fine, sandy, silty CLAY w/shells, wood frags., trace gravel.
Dark brownish gray, organic, gravelly, silty SAND w/shells & vegetation.
Gray, organic, gravelly, silty SAND w/ shells, trace gravel, organic odor.
Gray-brown, gravelly, sandy, silty CLAY (TILL).
Gray, gravelly, sandy, silty CLAY (TILL).
Core-drilled Rec. 0.4'
Core-Drilled, No Rec.
Gray, gravelly, clayey SAND (TILL).
Core-Drilled, Rec. 0.5'
Grey, sandy SILT.
Brownish-grey, gravelly, silty SAND.
Core-Drilled
Gray, silty, clayey M-F SAND.
Gray, sandy CLAY (TILL).
Core-Drilled, Weathered ROCK.
ARGILLITE, light gray, soft w/ intermittent very soft zones weathered to consistency of clay. Core in fragments up to 4".
EL. 18.9±
FD-3
15 NOV. 1970
EL. 119.4
Bituminous Concrete
Brown, silty, gravelly SAND w/ trace wood & pcs. asphalt.
Brown, silty, gravelly SAND w/ trace wood.
Brown, gravelly SAND (FILL)
No Recovery.
No Recovery
Gray, gravelly SAND and Black, silty SAND w/cinders, trace gravel, and pcs. wood (FILL)
Dark grey, gravelly SAND w/traces coal & silt.
Black, organic SILT w/shells, fibers, oil & organic odor.
Black, organic SILT w/shells, fibers, oil, cinders and organic odor.
Black, organic SILT w/shells & cinders.
Grey, organic, silty SAND w/shell frags. & occasional pc. gravel.
Dark gray, silty, sandy GRAVEL.
Grey brown to grey, silty, sandy GRAVEL w/occasional cobble.
Gray brown sandy CLAY (TILL)
Gray brown, gravelly, sandy CLAY w/ occasional pc. large gravel. (TILL)
No Sample
Core-drilled "COBBLE, GRAVEL & TILL"
Gray-brown, sandy, clayey GRAVEL (TILL)
Gray-brown, sandy, gravelly CLAY (TILL)
Weathered COBBLE
Core-drilled "GRAVEL & TILL" Run 4.2' Rec. 0.5'
Weathered ROCK
ARGILLITE, light gray, massive to slaty in part. Occ. very soft zones, core in small fragments.
DEPTH IN FEET
SCALE IN FEET
1" = 5'
LEGEND FOR GRAPHIC LOGS
FD — Foundation Test Boring
25 NOV. 1970 — Date exploration completed.
EL. 123.8 — Elevation of ground surface during time of exploration.
7.0' — Depth of water at time of exploration
Subsurface water level in boring at time of exploration.
SM — Group letter symbol according to Unified Soil Classification System.
NR — No Recovery or unsatisfactory soils samples recovered.
NS — Not Sampled. Hole advanced by Core-drilling, blasting and/or wash boring due to operational difficulty
CD — Sampling in overburden by Core-drilled method.
25 — Blows per foot of penetration using a 350 pound hammer with a free fall of about 30" on a 1⅜" I.D. and 2" O.D. or 2" I.D. and 2½" O.D. Size 5-foot solid sample spoon equipped with a beveled and sharpened drive shoe.
* — Blow counts not recorded or not considered representative.
50% — Percent of rock core recovery using double tube core barrels to recover 1⅝ or 2⅛ inch diameter cores.
NOTES:
For Notes see Plate 4-2
REVISION DATE DESCRIPTION BY
CHARLES A. MAGUIRE AND ASSOCIATES INC. WALTHAM, MASSACHUSETTS
DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION, CORPS OF ENGINEERS WALTHAM MASSACHUSETTS
WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
RECORD OF SUBSURFACE EXPLORATIONS NO. 8
CHARLES RIVER BASIN MASSACHUSETTS
SCALE: 1" = 5'
SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 4-10



NOTES:
For Notes see Plate 4-2

SCALE IN FEET
1"=5' 5' 0 5' 10'

REVISION	DATE	DESCRIPTION	BY

CHARLES A. MAGUIRE AND ASSOCIATES INC.
WALTHAM, MASSACHUSETTS

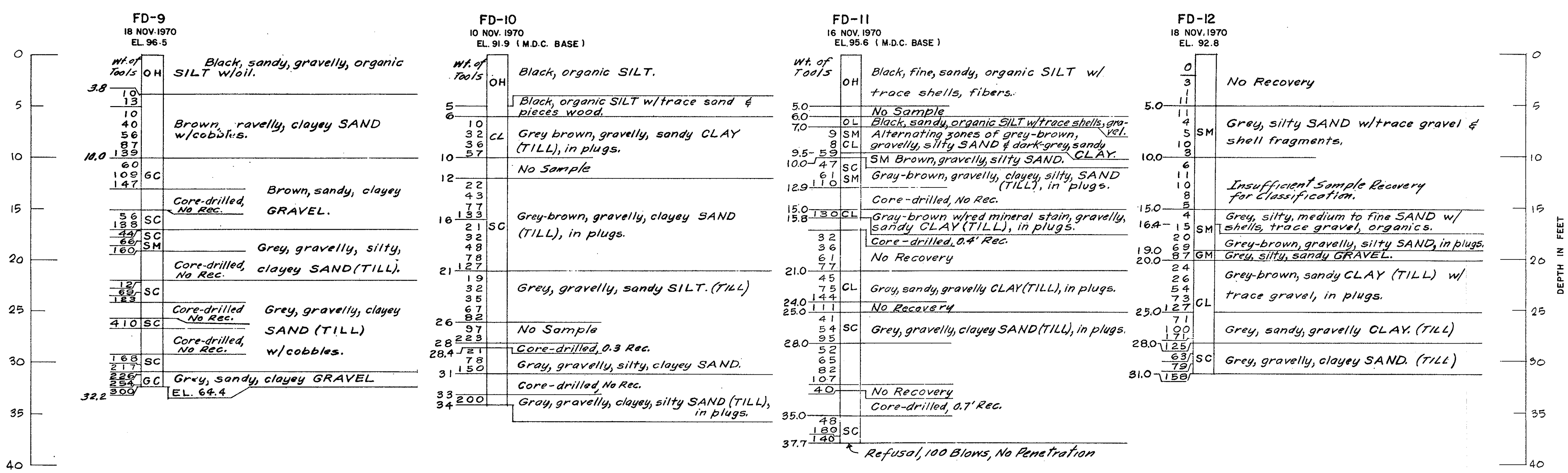
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM MASSACHUSETTS

DES. BY | DR. BY | CK. BY
SUBMITTED:
ARCHITECT - ENGINEER
APPROVAL RECOMMENDED:
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
RECORD OF SUBSURFACE
EXPLORATIONS NO. 9
CHARLES RIVER BASIN MASSACHUSETTS

APPROVED DATE
CHIEF ENGINEERING DIVISION
SCALE: 1"=5' SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 4-11



NOTES:
For Notes see Plate 4-2

SCALE IN FEET
1" = 5' 5' 0 5' 10'

REVISION	DATE	DESCRIPTION	BY

CHARLES A. MAGUIRE AND ASSOCIATES INC. WALTHAM, MASSACHUSETTS

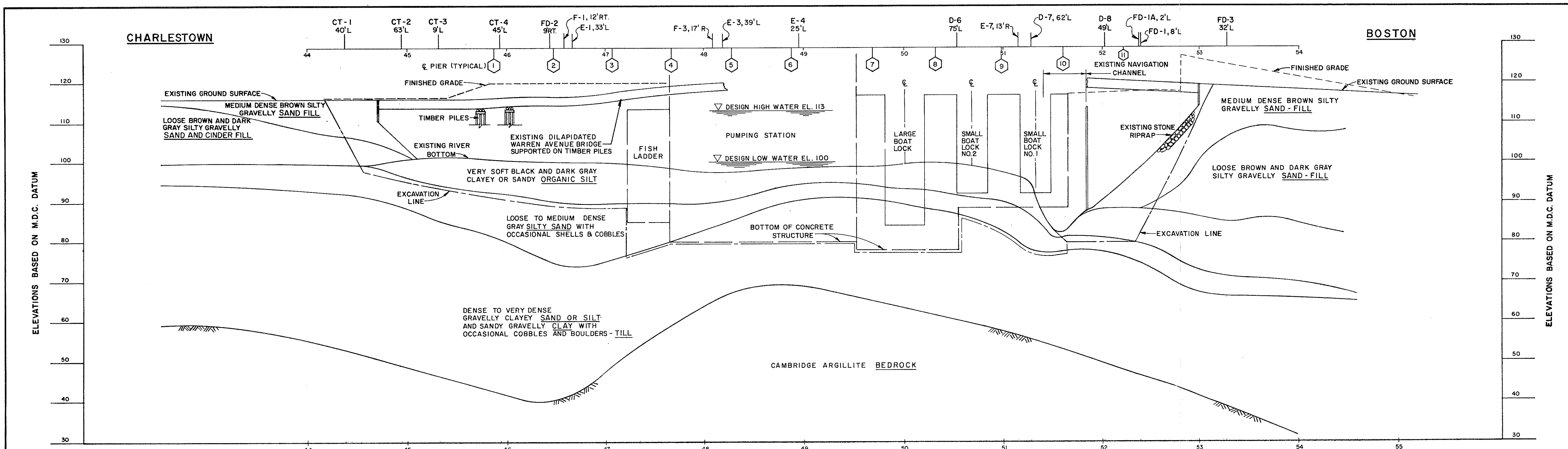
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM MASSACHUSETTS

DES. BY | DR. BY | CK. BY
SUBMITTED:
ARCHITECT - ENGINEER
APPROVAL RECOMMENDED:
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
RECORD OF SUBSURFACE EXPLORATIONS NO. 10
CHARLES RIVER BASIN MASSACHUSETTS

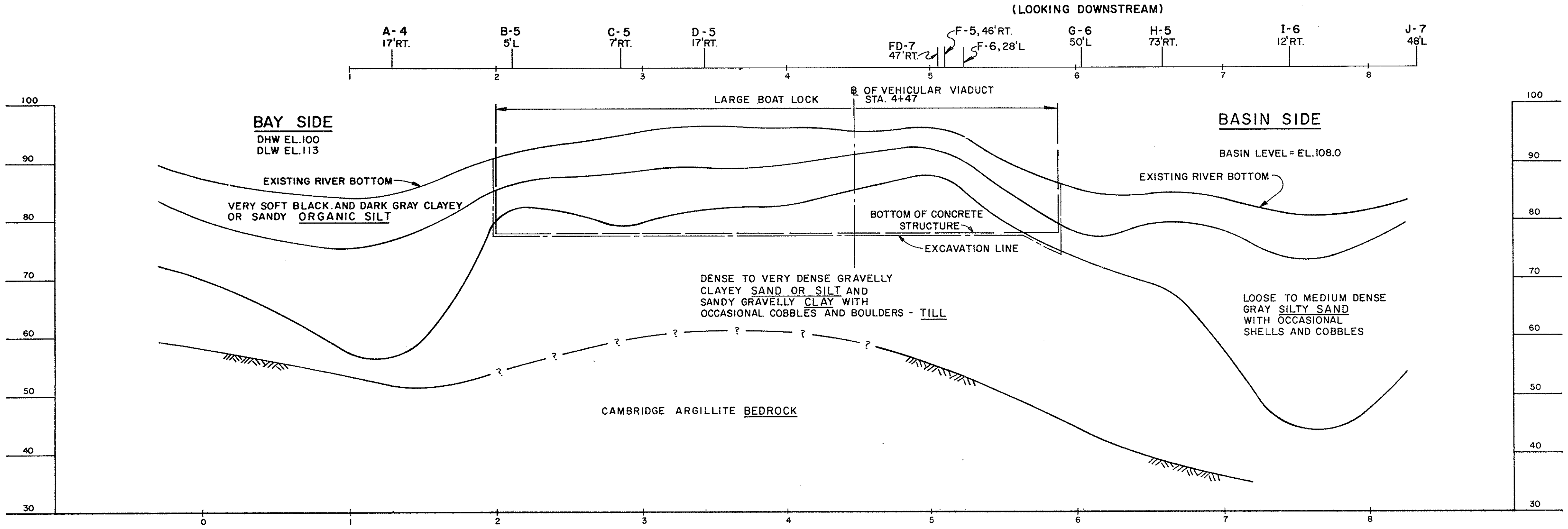
APPROVED DATE
CHIEF ENGINEERING DIVISION
SCALE: 1" = 5' | SPEC. NO.
DRAWING NUMBER
SHEET

PLATE 4-12



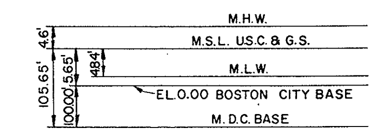
GENERALIZED SOIL PROFILE ALONG BASELINE OF VEHICULAR VIADUCT

(LOOKING DOWNSTREAM)



GENERALIZED SOIL PROFILE ALONG CENTERLINE OF LARGE LOCK

(LOOKING SOUTH)



DATUM RELATIONSHIP

SCALE IN FEET

VERT. 0 10 20 30 40 50

HOR. 0 40 80 120 160 200

WATER RESOURCES DEVELOPMENT PROJECT

CHARLES RIVER LOCKS AND DAM

CHARLES RIVER BASIN MASSACHUSETTS

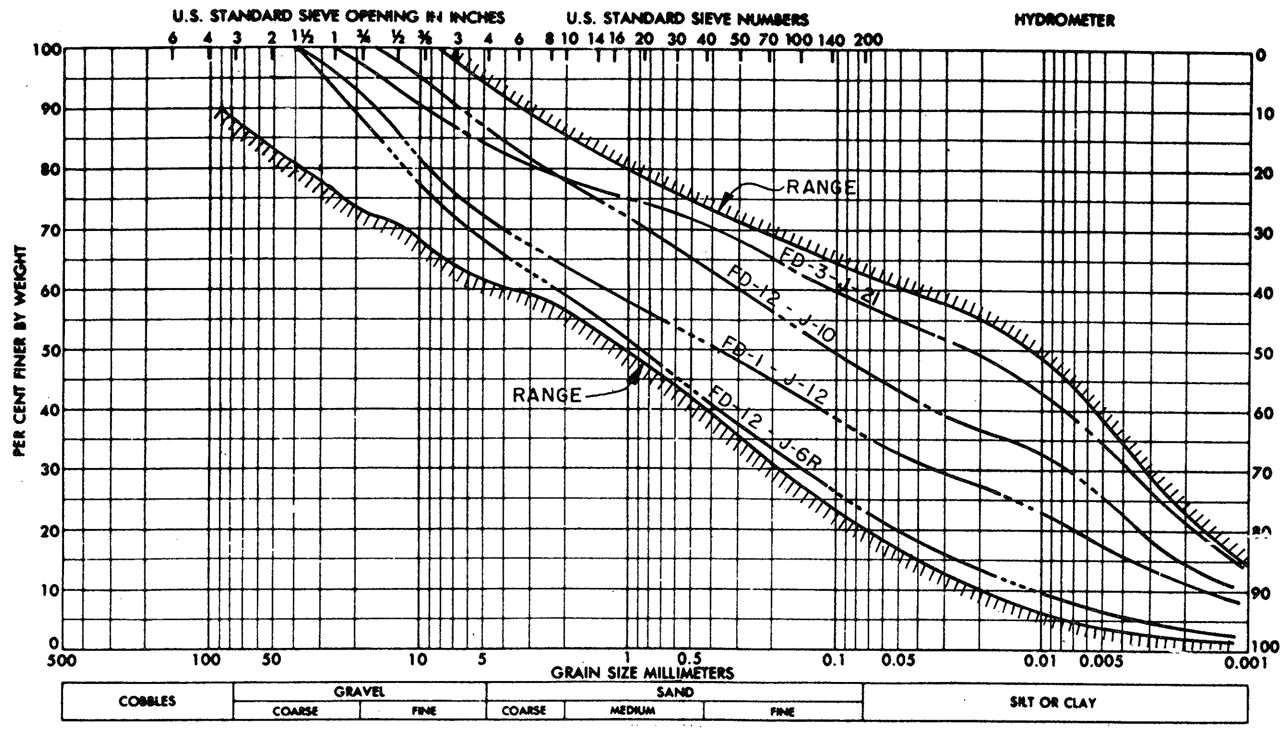
GENERALIZED SOIL PROFILES

DEPARTMENT OF THE ARMY

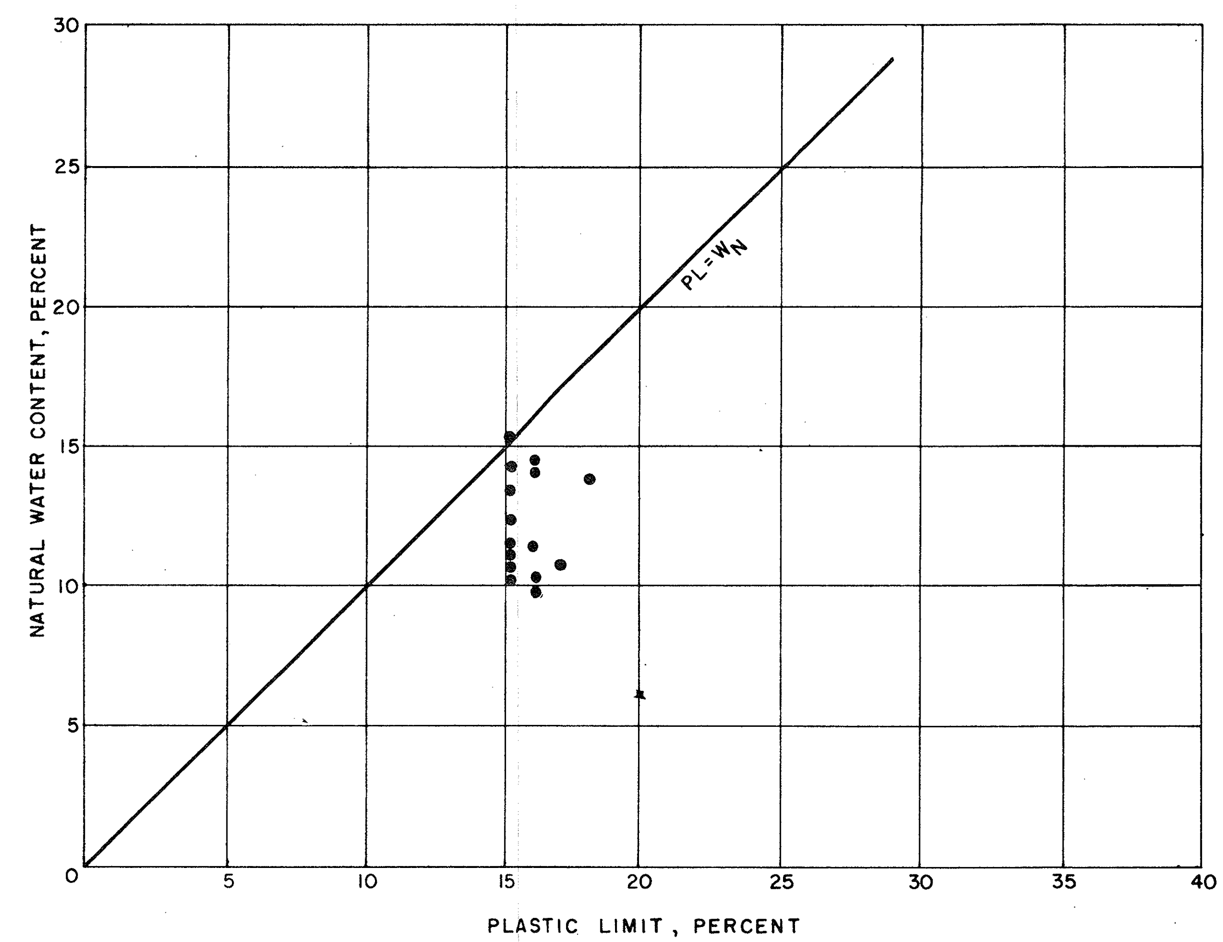
NEW ENGLAND DIVISION

CORPS OF ENGINEERS WALTHAM, MASS.

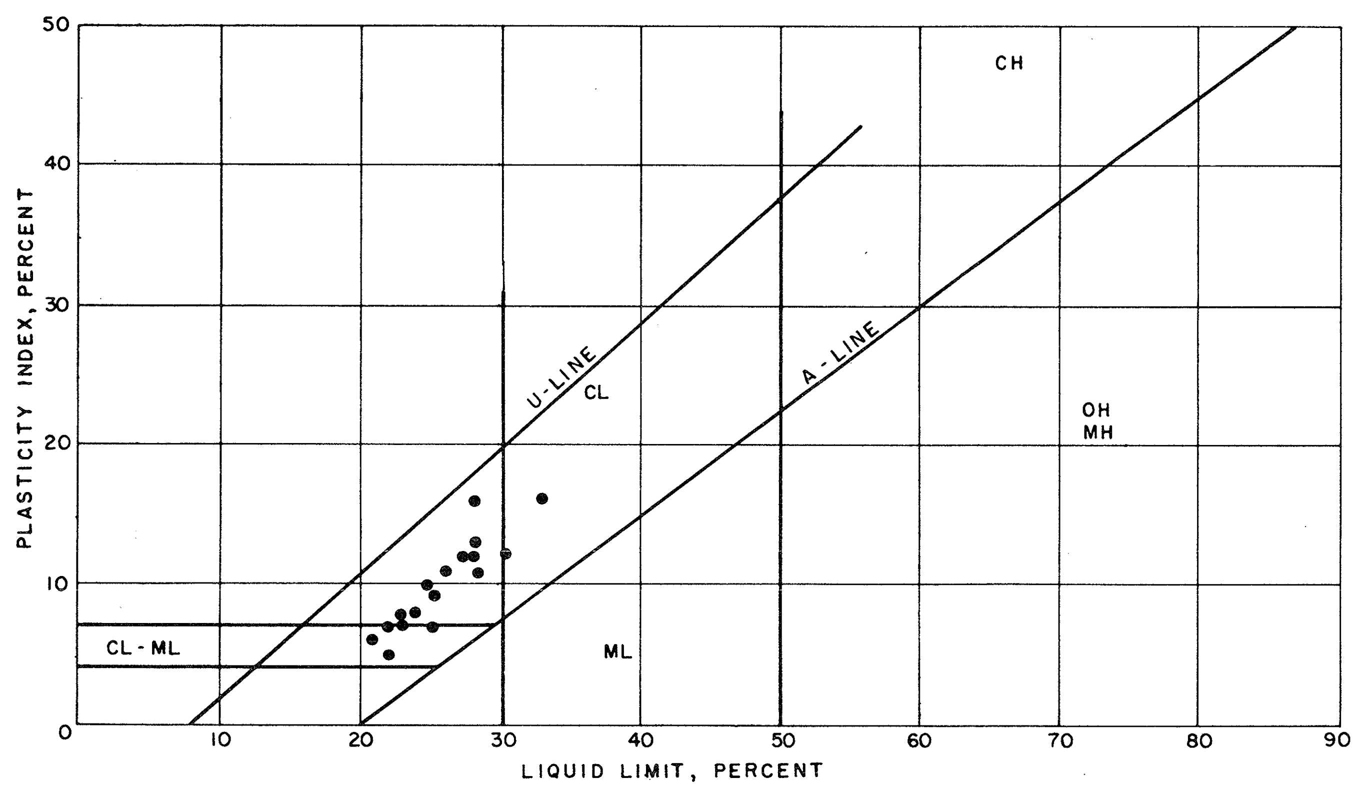
PLATE 4-13



GRADATION CURVES



PLASTIC LIMIT VS. NATURAL WATER CONTENT



PLASTICITY CHART

WATER RESOURCES DEVELOPMENT PROJECT

CHARLES RIVER LOCKS AND DAM

CHARLES RIVER BASIN MASSACHUSETTS

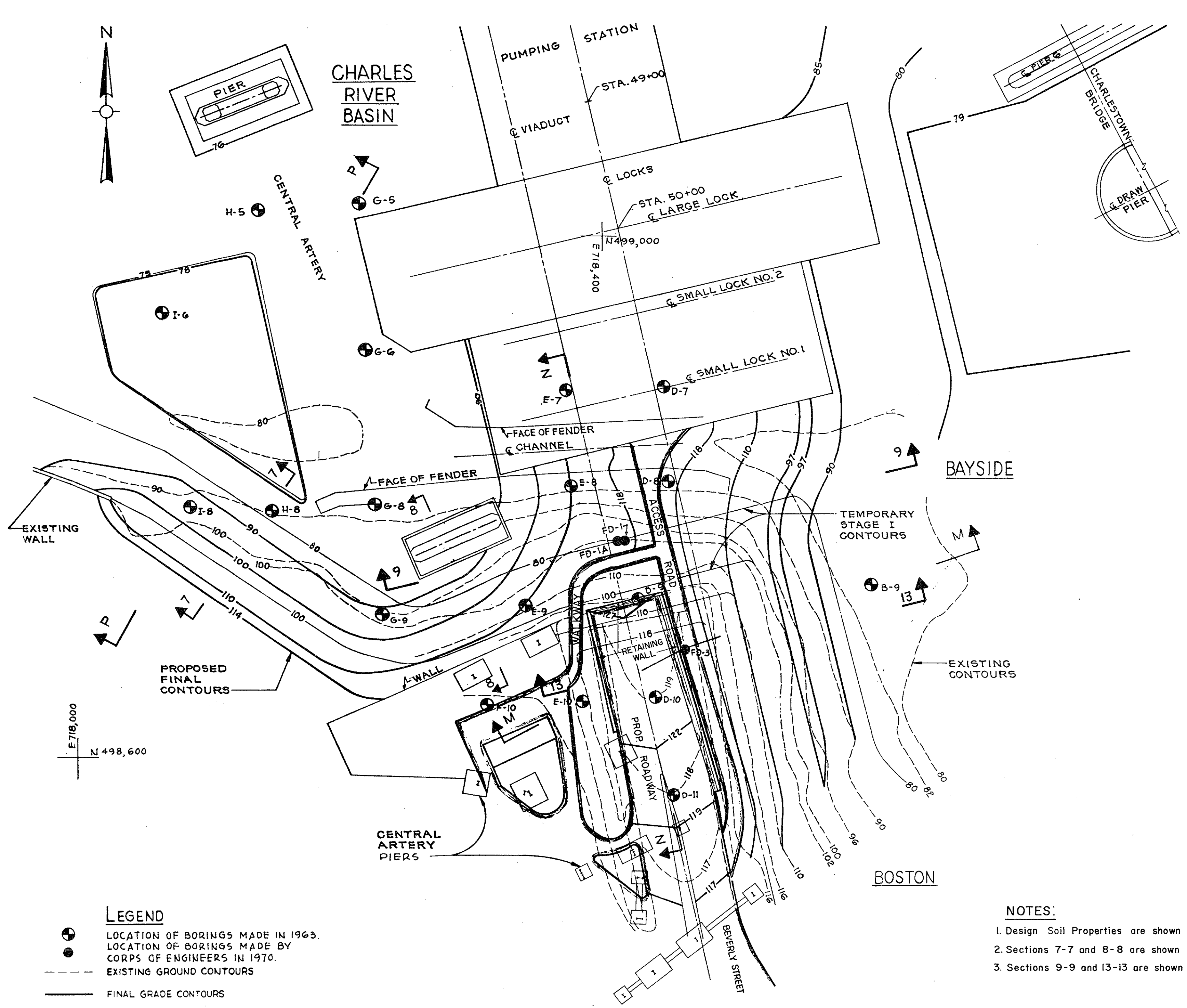
SELECTED TEST DATA

TILL

DEPARTMENT OF THE ARMY

NEW ENGLAND DIVISION

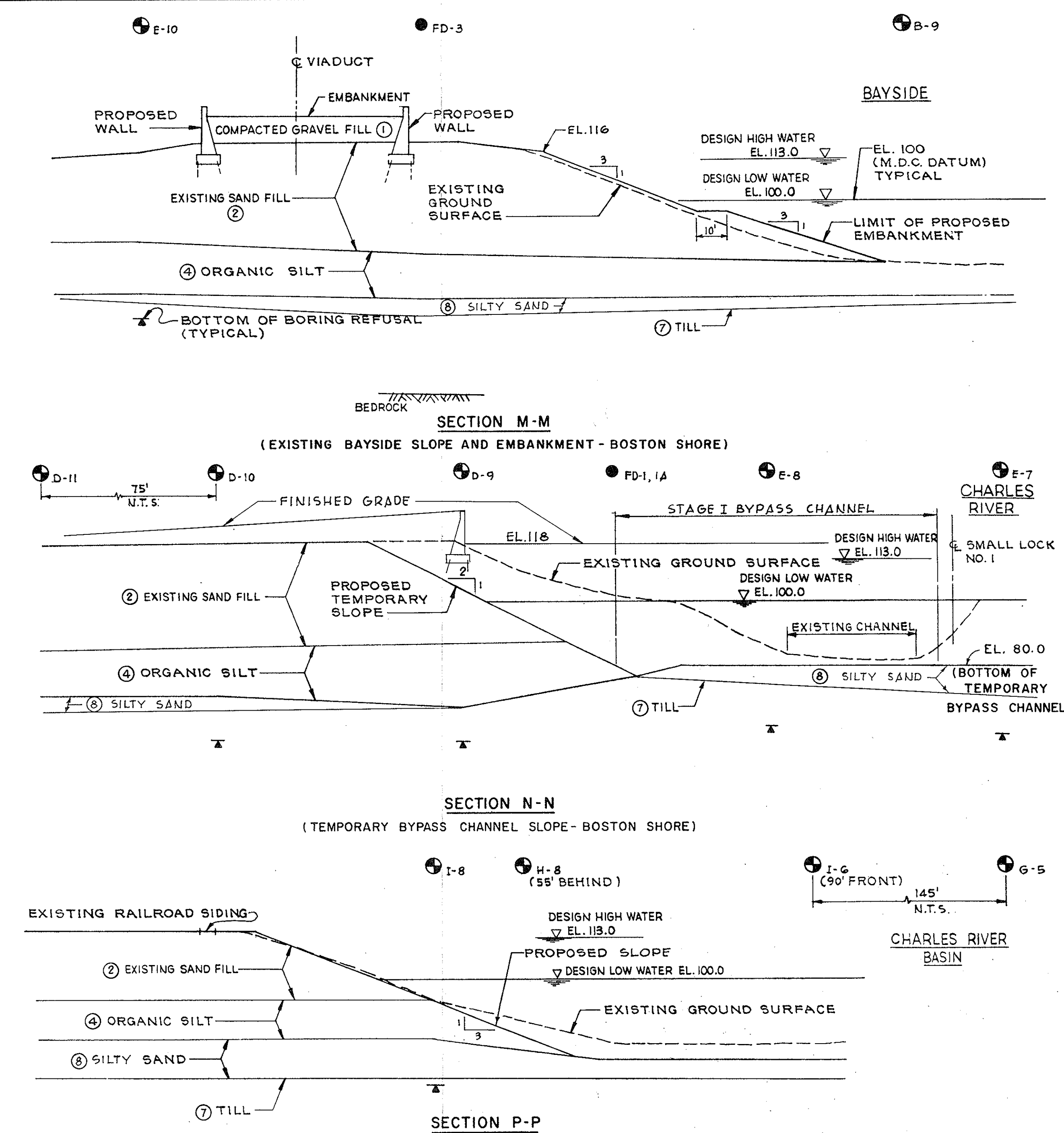
CORPS OF ENGINEERS WALTHAM, MASS.



PLAN

SCALE FOR PLAN ONLY

0 40 80 120



SECTION M-M
(EXISTING BAYSIDE SLOPE AND EMBANKMENT - BOSTON SHORE)

SECTION N-N
(TEMPORARY BYPASS CHANNEL SLOPE - BOSTON SHORE)

SECTION P-P
(BASIN SIDE SLOPE – BOSTON SHORE)

SCALE FOR SECTIONS ONLY

0 20 40 60

LEGEND

- LOCATION OF BORINGS MADE IN 1963.
- LOCATION OF BORINGS MADE BY CORPS OF ENGINEERS IN 1970.
- EXISTING GROUND CONTOURS
- FINAL GRADE CONTOURS
- TEMPORARY CONSTRUCTION STAGE I CONTOURS
- P P LOCATION OF SECTION P-P
- ① SOIL IDENTIFICATION NUMBER

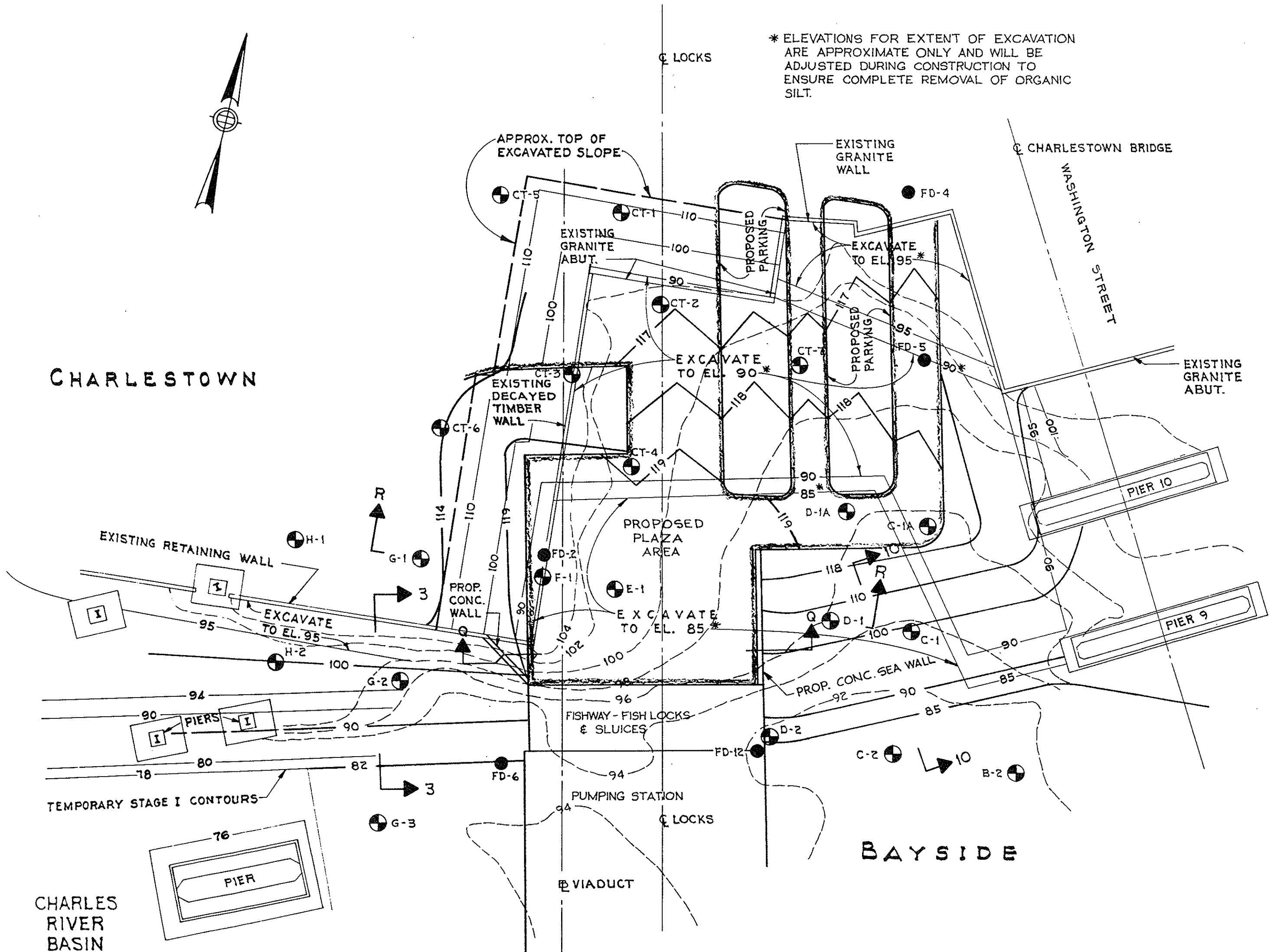
NOTES:

1. Design Soil Properties are shown on PLATE 4-17
2. Sections 7-7 and 8-8 are shown on PLATE 4-21
3. Sections 9-9 and 13-13 are shown on PLATE 4-22

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
CHARLES RIVER BASIN MASSACHUSETTS

TYPICAL EMBANKMENT AND SLOPE
PLAN AND SECTIONS

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS WALTHAM, MASS.

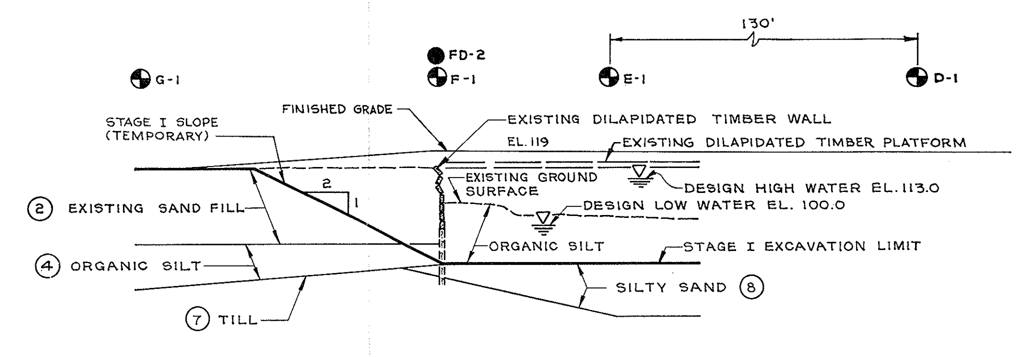


PLAN

SCALE FOR PLAN ONLY
0 40 80 120

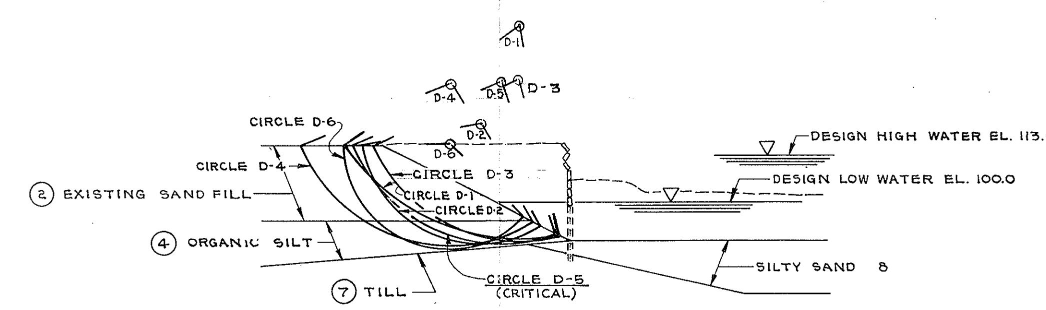
LEGEND

- LOCATION OF BORING MADE IN 1963
- LOCATION OF BORING MADE BY CORPS OF ENGINEERS IN 1970
- EXISTING GROUND CONTOURS
- FINAL GRADE CONTOURS
- TEMPORARY CONSTRUCTION STAGE I CONTOURS
- R R LOCATION OF SECTION R-R
- ① SOIL IDENTIFICATION NUMBER



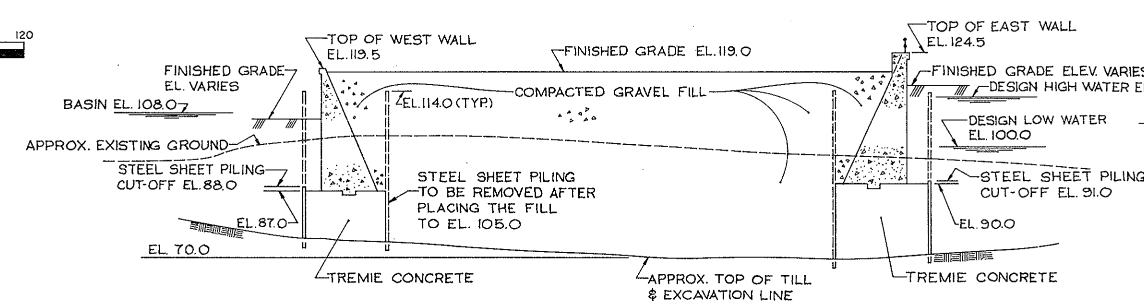
TEMPORARY EXCAVATION SLOPE - CHARLESTOWN SHORE - SECTION R-R

NOTES:
1. DESIGN SOIL PROPERTIES ARE SHOWN ON PLATE 4-17
2. SECTIONS 3-3 IS SHOWN ON PLATE 4-21
3. SECTION 10-10 IS SHOWN ON PLATE 4-22



TEMPORARY EXCAVATION SLOPE AT LOW TIDE - CHARLESTOWN SHORE - SECTION R-R

SCALE FOR SECTIONS ONLY
0 20 40 60



SECTION Q-Q

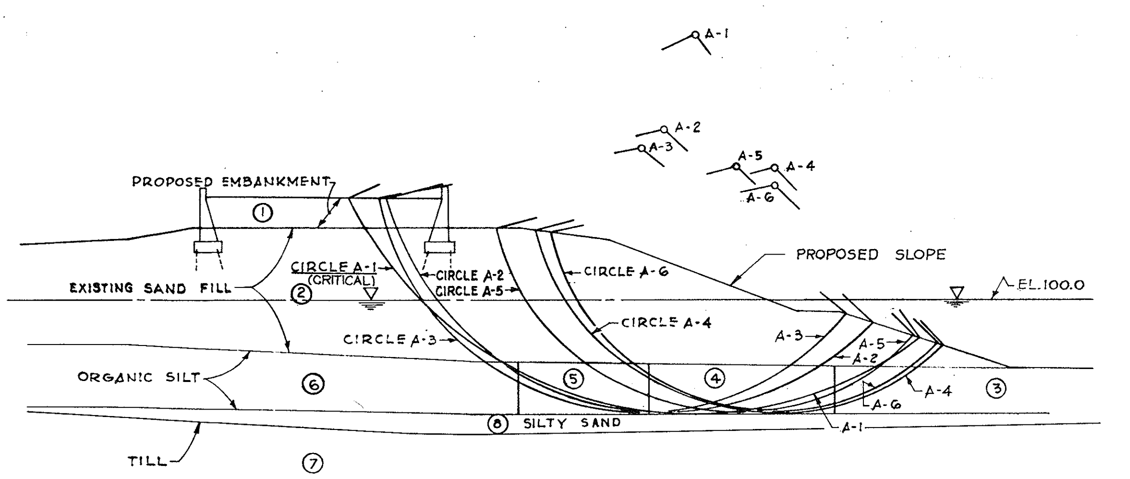
SCALE FOR SECTION ONLY
0 20 40 60

COMPACTED GRAVEL FILL
EL. 105
DUMPED GRAVEL FILL

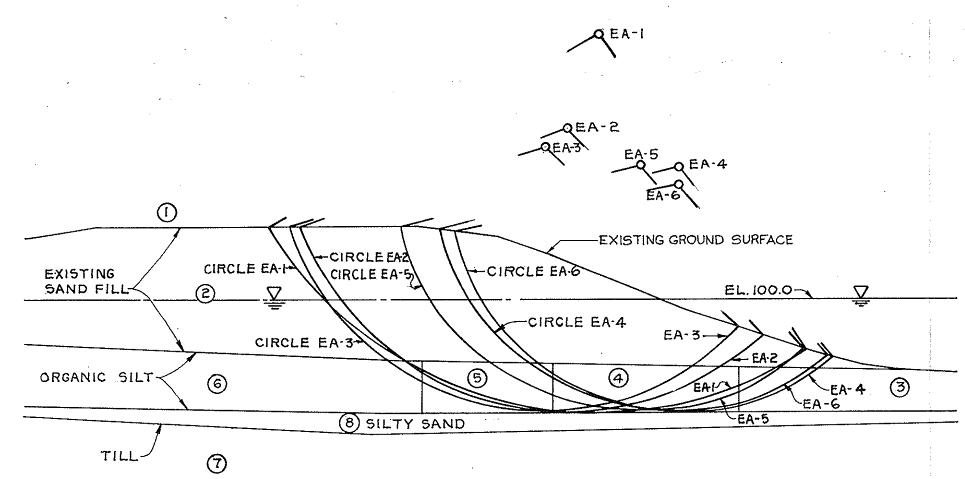
WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
CHARLES RIVER BASIN MASSACHUSETTS

TYPICAL EXCAVATION SLOPE - CHARLESTOWN SHORE
PLAN AND SECTIONS

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS WALTHAM, MASS.

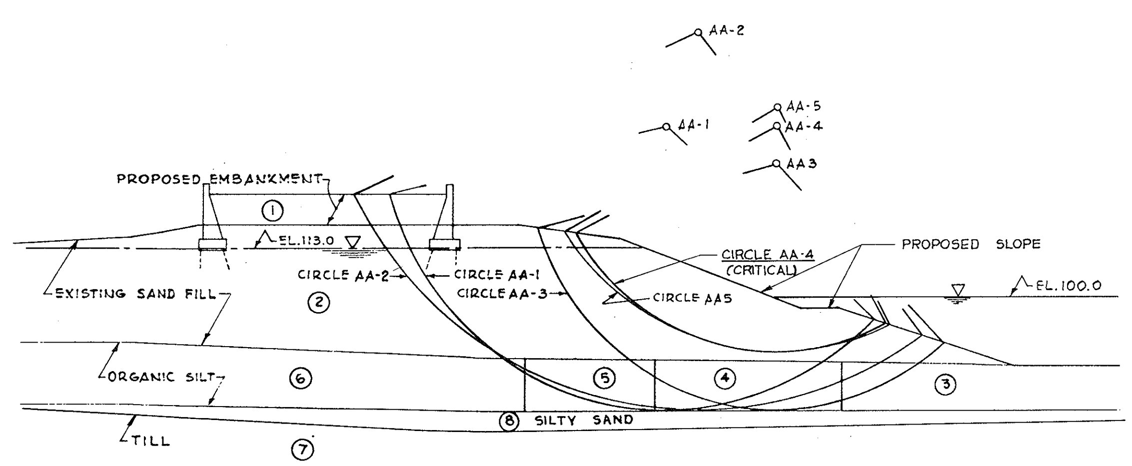


BOSTON EMBANKMENT-LOW TIDE ANALYSIS-SECTION M-M
(BAYSIDE)

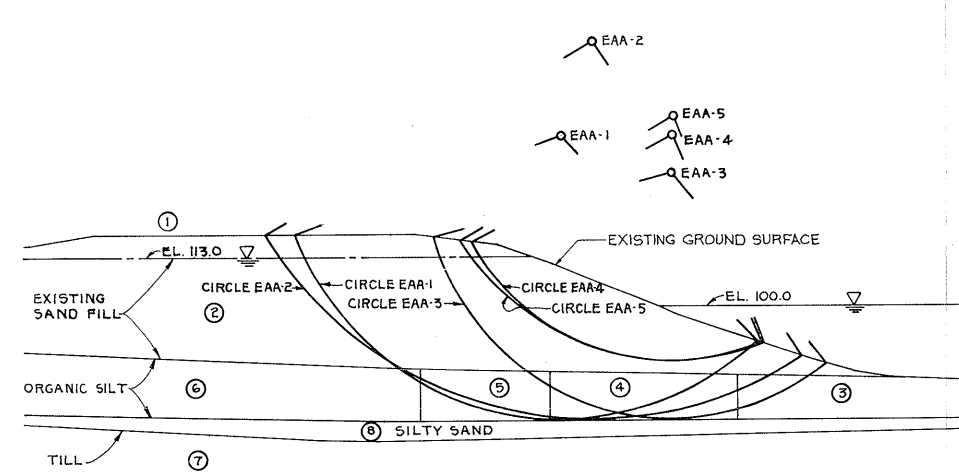


EXISTING SLOPE-LOW TIDE ANALYSIS-SECTION M-M
(BOSTON SHORE-BAYSIDE)

DESIGN VALUES										
SOIL NO.	MATERIAL	UNIT WEIGHT IN POUNDS PER CUBIC FOOT			SHEAR STRENGTH					
					S-CONDITION		Q-CONDITION		R-CONDITION	
		MOIST	SATURATED	SUBMERGED	ϕ IN DEGREES	C (in T/SF)	ϕ IN DEGREES	C (in T/SF)	ϕ IN DEGREES	C (in T/SF)
1	COMPACTED AND DUMPED GRAVEL FILLS	135	135	71	35	0	35	0	35	0
2	EXISTING SAND FILL	110	110	47.6	33	0	33	0	33	0
3	ORGANIC SILT	95	95	32.6	36	0	-	0.15	14	0.10
4	ORGANIC SILT	95	95	32.6	36	0	-	0.30	14	0.10
5	ORGANIC SILT	95	95	32.6	36	0	-	0.40	14	0.10
6	ORGANIC SILT	95	95	32.6	36	0	-	0.50	14	0.10
7	TILL	-	-	-	-	-	-	-	-	-
8	SILTY SAND	110	110	47.6	33	0	33	0	33	0



BOSTON EMBANKMENT-RAPID DRAWDOWN ANALYSIS FROM HIGH TIDE-SECTION M-M
(BAYSIDE)



EXISTING SLOPE-RAPID DRAWDOWN ANALYSIS FROM HIGH TIDE-SECTION M-M
(BOSTON SHORE-BAYSIDE)

SCALE IN FEET
0 20 50 100

NOTES:

1. ADDITIONAL INFORMATION AND LOCATIONS OF SECTIONS ARE SHOWN ON PLATES 4-15, 4-16, 4-18, 4-19 AND 4-20
2. LOCATION OF SECTION M-N IS SHOWN ON PLATE 4-15

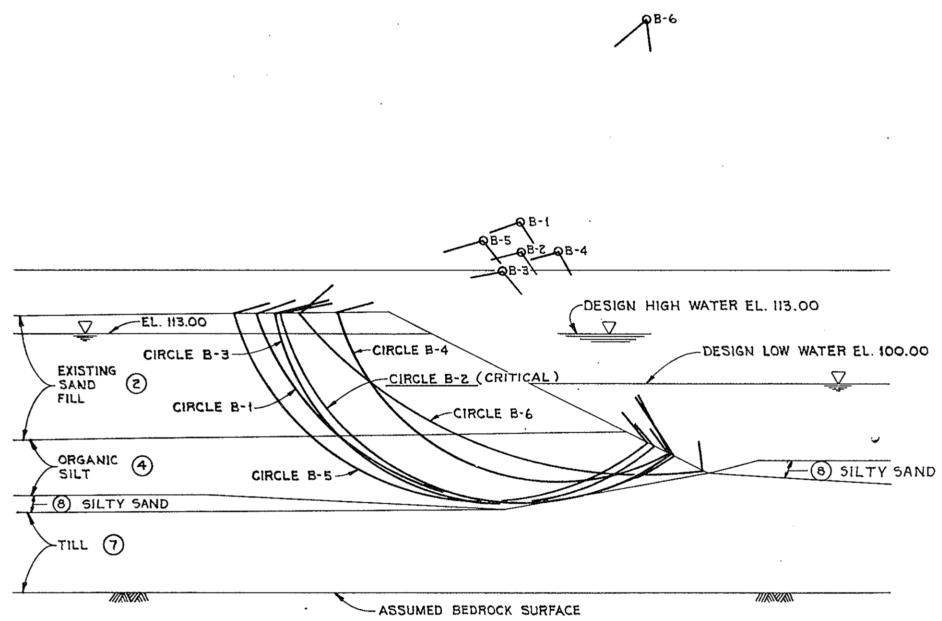
SUMMARY OF STABILITY ANALYSIS			
CONDITION ANALYZED	PORE PRESSURE ASSUMPTION	ARC	COMPUTED FACTOR OF SAFETY
a. TEMPORARY EXCAVATION SLOPES (1) BYPASS CHANNEL SLOPE (BOSTON SHORE) SECTION N-N, PLATE 4-18 (a) CIRCLE ANALYSIS (I) LOW TIDE	(1)	B-1	1.43
		B-2	1.42 MIN.
		B-3	1.46
		B-4	1.50
		B-5	1.51
		B-6	1.76
(II) RAPID DRAWDOWN	(2)	BB-1	1.33 MIN.
		BB-2	1.36
		BB-3	1.38
		BB-4	1.34
		BB-5	1.37
(b) WEDGE ANALYSIS (1) LOW TIDE	(1)	-	1.29
(2) RAPID DRAWDOWN	(2)	-	1.10
(2) BASIN SIDE SLOPE (BOSTON SHORE) SECTION P-P, PLATE 4-18 (a) LOW TIDE (I) CIRCLE ANALYSIS	(1)	C-1	1.52 MIN.
		C-2	1.52
		C-3	1.52
		C-4	1.53
		C-5	1.57
		C-6	2.16
(b) RAPID DRAWDOWN (I) CIRCLE ANALYSIS	(2)	CC-1	1.34 MIN.
		CC-2	1.38
		CC-3	1.39
		CC-4	1.36
		CC-5	1.35
		CC-6	1.90
(3) SLOPE AFTER DREDGING (CHARLESTOWN SHORE) SECTION R-R, PLATE 4-16 (a) LOW TIDE (I) CIRCLE ANALYSIS	(1)	D-1	1.54
		D-2	1.57
		D-3	1.59
		D-4	1.77
		D-5	1.42 MIN.
		D-6	1.57
b. EMBANKMENT (1) BOSTON EMBANKMENT SECTION M-M, PLATE 4-17 (a) LOW TIDE (I) CIRCLE ANALYSIS	(1)	A-1	1.33 MIN.
		A-2	1.36
		A-3	1.35
		A-4	1.34
		A-5	1.39
		A-6	1.37
(b) RAPID DRAWDOWN (I) CIRCLE ANALYSIS	(2)	AA-1	1.29
		AA-2	1.20
		AA-3	1.24
		AA-4	1.15 MIN.
		AA-5	1.25
c. EXISTING SLOPE (I) BAYSIDE SLOPE BOSTON SHORE SECTION M-M, PLATE 4-17 (a) LOW TIDE (I) CIRCLE ANALYSIS	(1)	EA-1	1.46
		EA-2	1.50
		EA-3	1.56
		EA-4	1.25 MIN.
		EA-5	1.30
		EA-6	1.27
(b) RAPID DRAWDOWN (I) CIRCLE ANALYSIS	(2)	EAA-1	1.36
		EAA-2	1.31
		EAA-3	1.15
		EAA-4	1.10 MIN.
		EAA-5	1.12

(1) SUBMERGED (BUOYANT) WEIGHTS USED BELOW WATER ELEVATION.
(2) SATURATED WEIGHTS USED FOR DRIVING FORCES AND SUBMERGED (BUOYANT) WEIGHTS USED FOR RESISTING FORCES.

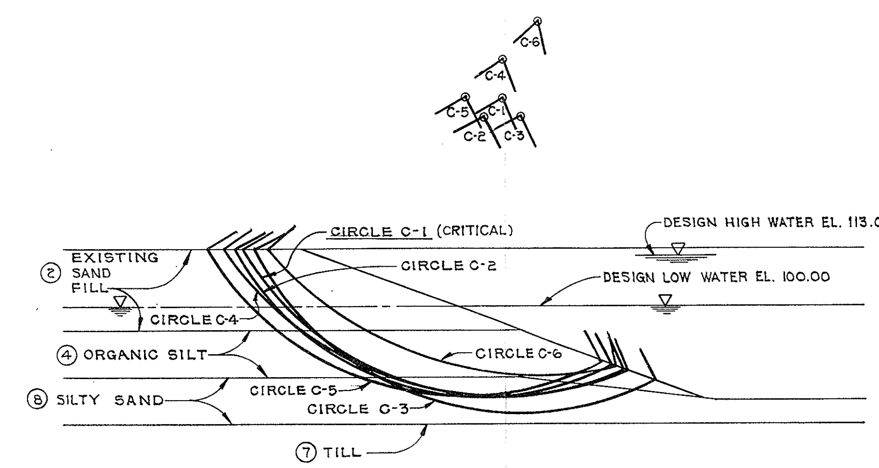
WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
CHARLES RIVER BASIN MASSACHUSETTS

SUMMARY OF STABILITY ANALYSES

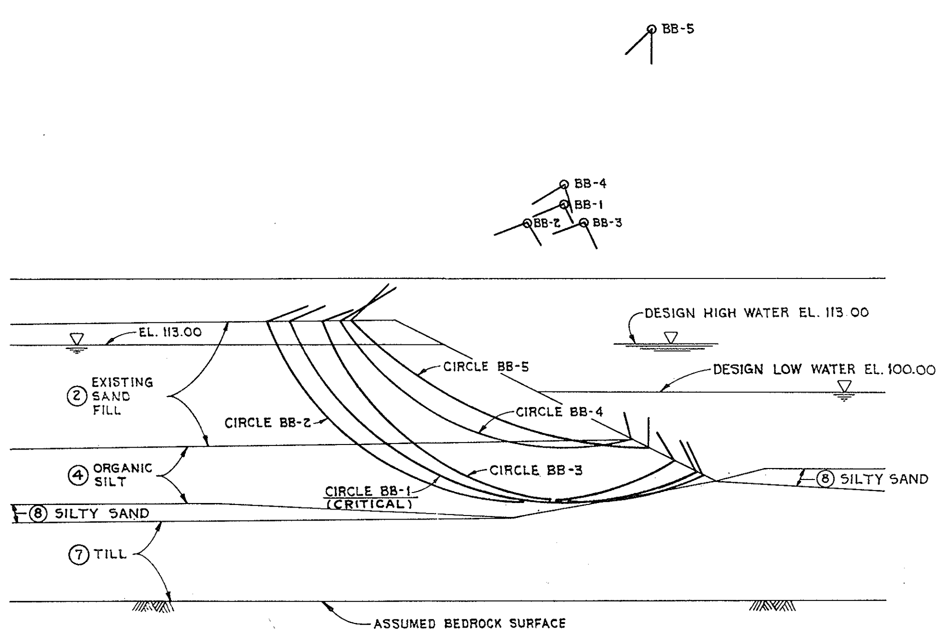
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS WALTHAM, MASS.



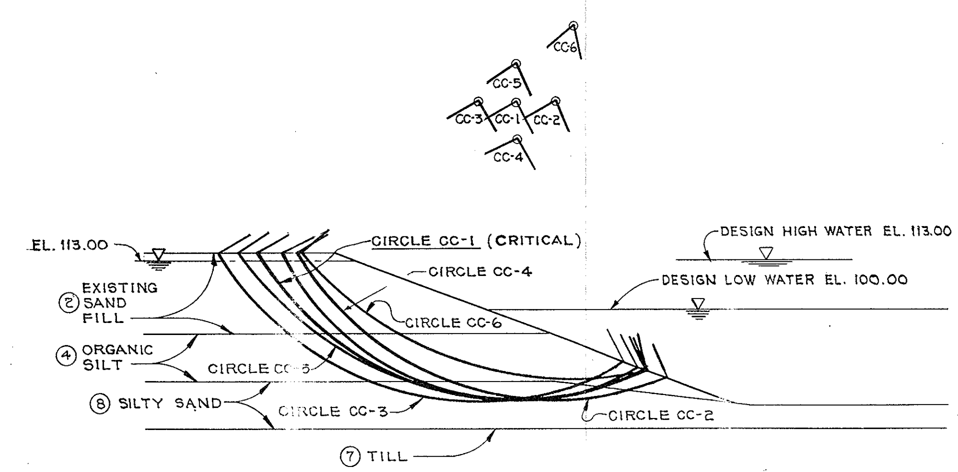
TEMPORARY BYPASS CHANNEL SLOPE AT LOW TIDE–SECTION N-N
(BOSTON SHORE)



TEMPORARY EXCAVATION SLOPE AT LOW TIDE - SECTION P-P
(BOSTON SHORE-BASIN SIDE)



TEMPORARY BYPASS CHANNEL SLOPE RAPID DRAWDOWN ANALYSIS-SECTION N-N
(BOSTON SHORE)



TEMPORARY EXCAVATION SLOPE RAPID DRAWDOWN ANALYSIS-SECTION P-P
(BOSTON SHORE- BASIN SIDE)

NOTES:

1. SUMMARY OF STABILITY ANALYSES IS SHOWN ON PLATE 4-17
2. DESIGN SOIL PROPERTIES ARE SHOWN ON PLATE 4-17
3. LOCATIONS OF SECTIONS N-N AND P-P ARE SHOWN ON PLATE 4-15

SCALE FOR SECTIONS ONLY
0 20 40 60

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
CHARLES RIVER BASIN MASSACHUSETTS

SUMMARY OF STABILITY ANALYSES

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS WALTHAM, MASS.

DESIGN VALUES					
SOIL NO.	MATERIAL	UNIT WEIGHT IN POUNDS PER CUBIC FOOT		SHEAR STRENGTH	
		MOIST	SUBMERGED	ϕ IN DEGREES	C IN tsf
1	GRAVEL FILL	135	71	35	—
2	EXISTING SAND FILL	110	47.6	33	—
3	ORGANIC SILT	95	32.6	—	0.15
4	ORGANIC SILT	95	32.6	—	0.30
5	ORGANIC SILT	95	32.6	—	0.40
6	ORGANIC SILT	95	32.6	—	0.50
7	TILL	140	76	38	0.15
8	SILTY SAND	110	47.6	33	—

FORCES			
SLICE	WEIGHT (KIPS)	TANGENTIAL FORCE (KIPS)	NORMAL FORCE (KIPS)
1	31.9	24.0	18.5
2	54.7	32.6	43.0
3	59.3	24.0	55.0
4	53.2	11.0	52.0
5	41.7	0.0	41.7
6	22.7	-4.0	22.0
7	14.1	-5.0	13.0
8	2.3	-1.5	2.0
	TOTAL	81.1	—

RESISTING FORCE $= \Sigma \bar{N} \tan\phi + \Sigma CL$

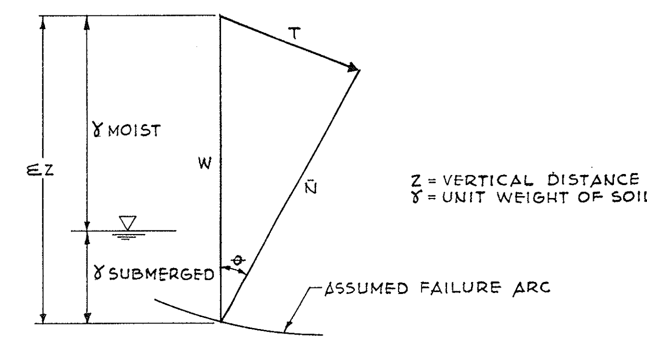
$$\Sigma \bar{N} \tan\phi = (18.5 + 43.0 + 2.0) \tan 33^\circ = 63.5 \times 0.65 = 41.3^K$$

$$\Sigma CL = 2 \times 1.00 + 36 \times 0.80 + 52 \times 0.60 + 15 \times 0.30 = 2.0 + 28.8 + 31.2 + 4.5 = 66.5\ K$$

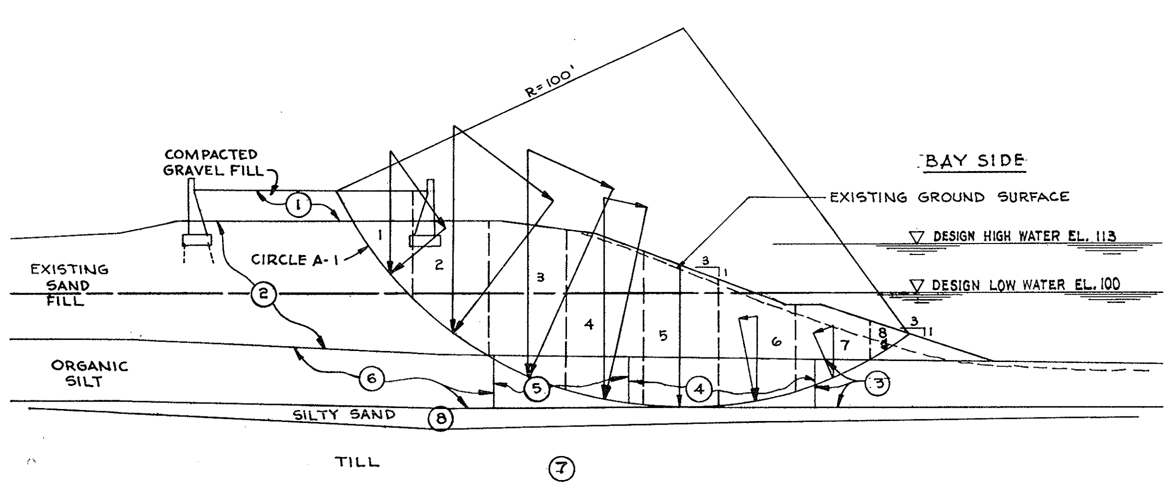
TOTAL RESISTING FORCE =

DRIVING FORCE = SUMMATION TANGENTIAL FORCES $= 81.1^K$

$$\text{FACTOR OF SAFETY} = \frac{\text{RESISTING FORCE}}{\text{DRIVING FORCE}} = \frac{107.8}{81.1} = 1.33$$

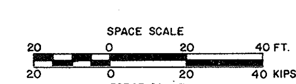


TYPICAL VECTOR DIAGRAM



SECTION M-M

BOSTON EMBANKMENT - CRITICAL CIRCLE - LOW TIDE ANALYSIS



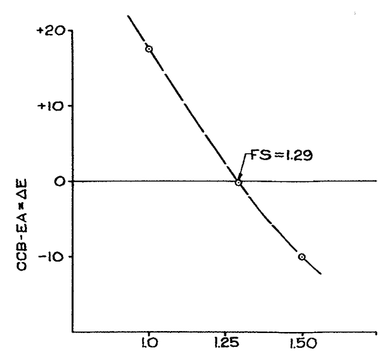
NOTES:

1. SEE PLATE 4-15 FOR SECTION LOCATION AND ADDITIONAL EMBANKMENT INFORMATION.
2. SUMMARY OF STABILITY ANALYSES IS SHOWN ON PLATE 4-17

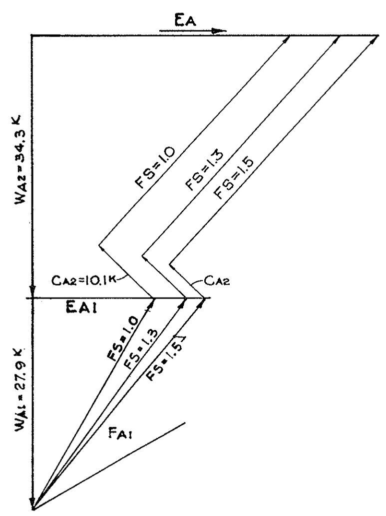
WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
CHARLES RIVER BASIN MASSACHUSETTS

TYPICAL STABILITY ANALYSIS
CRITICAL CIRCLE ANALYSIS

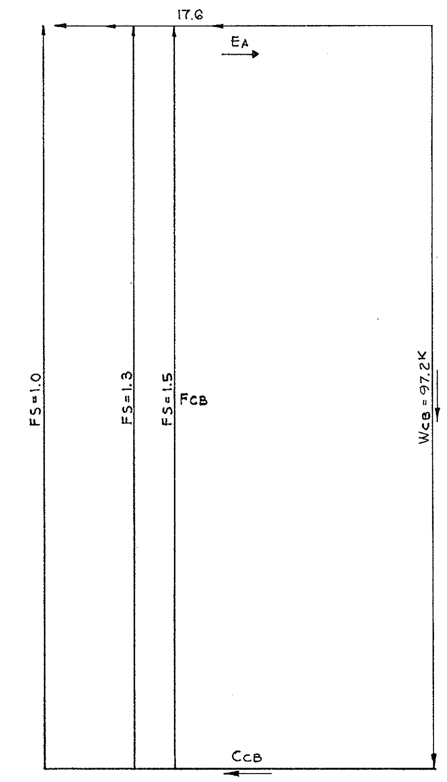
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS WALTHAM, MASS.



TRIAL FACTOR OF SAFETY, FS

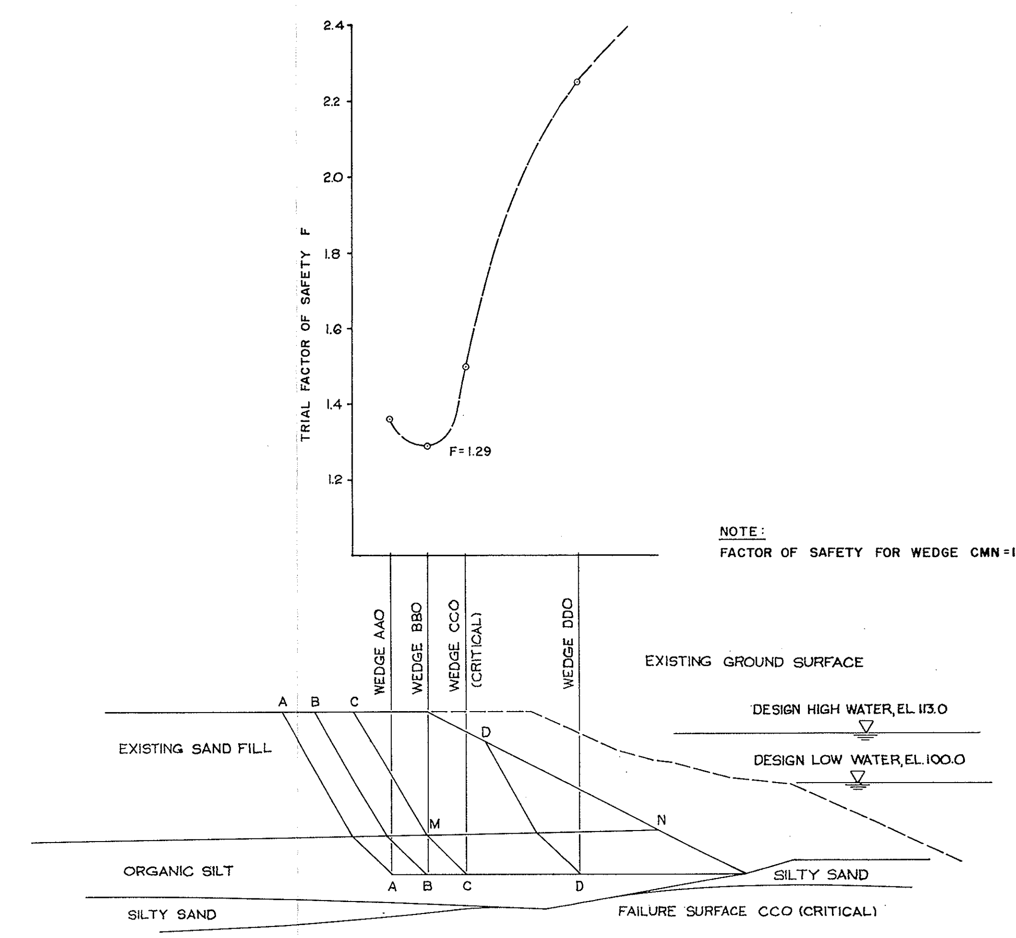


ACTIVE FORCE



RESISTING FORCE - NEUTRAL BLOCK

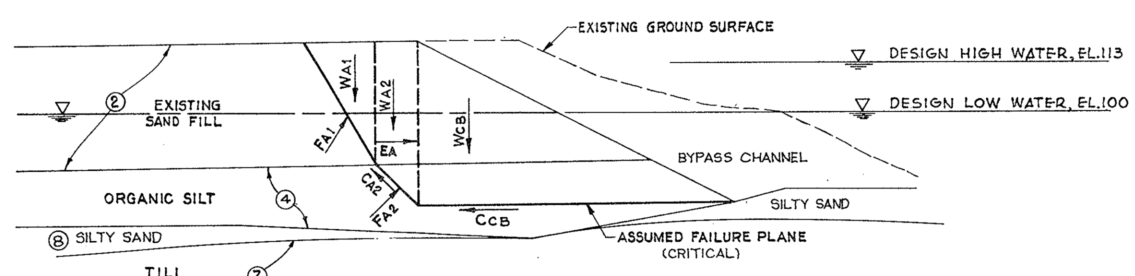
FS	F_{A1}	F_{A2}	C_{CB}	C_{A2}	E_A	ΔE
1.0	32.2^K	38.8^K	51.6^K	10.1^K	34.0^K	17.6^K
1.3	34.4	38.8^K	39.7^K	7.8^K	40.0^K	-0.3^K
1.5	35.9	38.8^K	34.4^K	6.7^K	45.0^K	-10.1^K



TEMPORARY BYPASS CHANNEL SLOPE - BOSTON SHORE - SECTION N-N

SUMMARY OF WEDGE ANALYSES

LOW TIDE ANALYSES



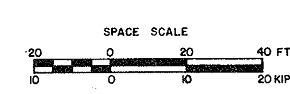
TEMPORARY BYPASS CHANNEL SLOPE - BOSTON SHORE

(SECTION N-N)

SOIL NO.	MATERIAL	Y WET pcf	Y BUOYANT pcf	Q SHEAR STRENGTH	
				ϕ	C
2	EXISTING SAND FILL	110	47.6	33°	—
4	ORGANIC SILT	95	32.6	0°	600 psf
7	TILL				

NOTES:

1. FOR LOCATION OF SECTION N-N AND ADDITIONAL INFORMATION SEE PLATE 4-15.
2. SUMMARY OF STABILITY ANALYSES ARE SHOWN ON PLATE 4-17.



WATER RESOURCES DEVELOPMENT PROJECT

CHARLES RIVER LOCKS AND DAM

CHARLES RIVER BASIN MASSACHUSETTS

TYPICAL STABILITY ANALYSIS

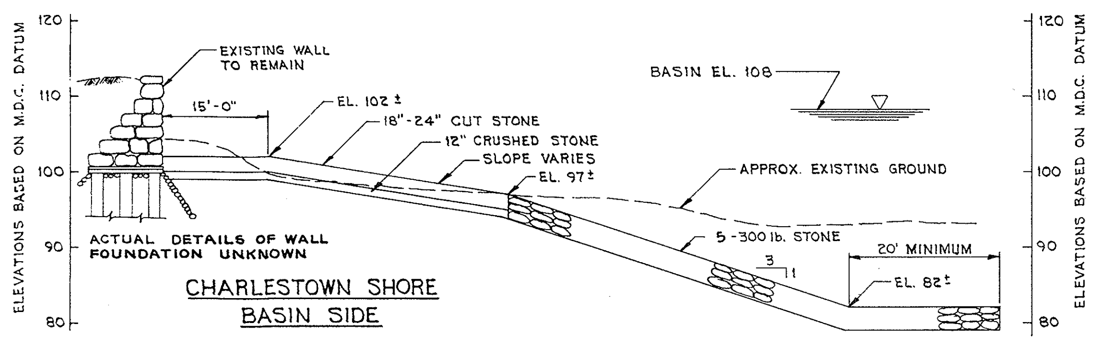
(WEDGE METHOD)

DEPARTMENT OF THE ARMY

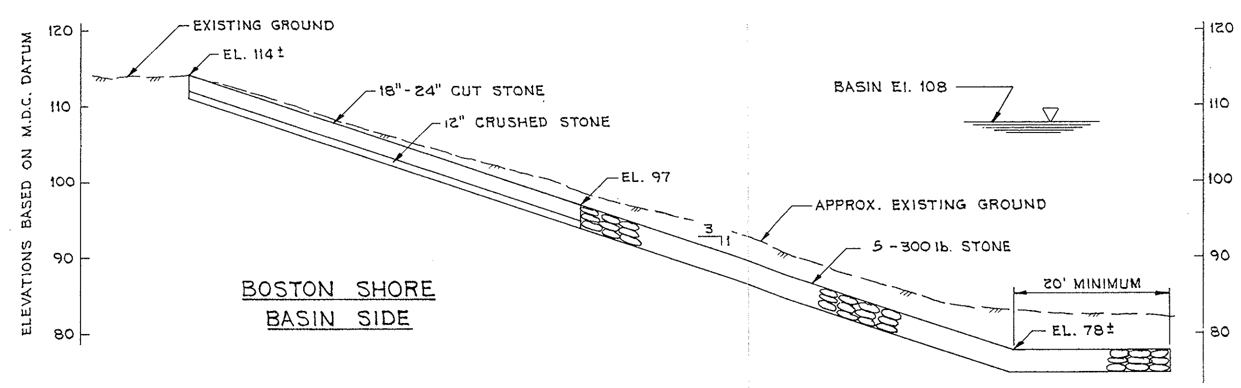
NEW ENGLAND DIVISION

CORPS OF ENGINEERS WALTHAM, MASS.

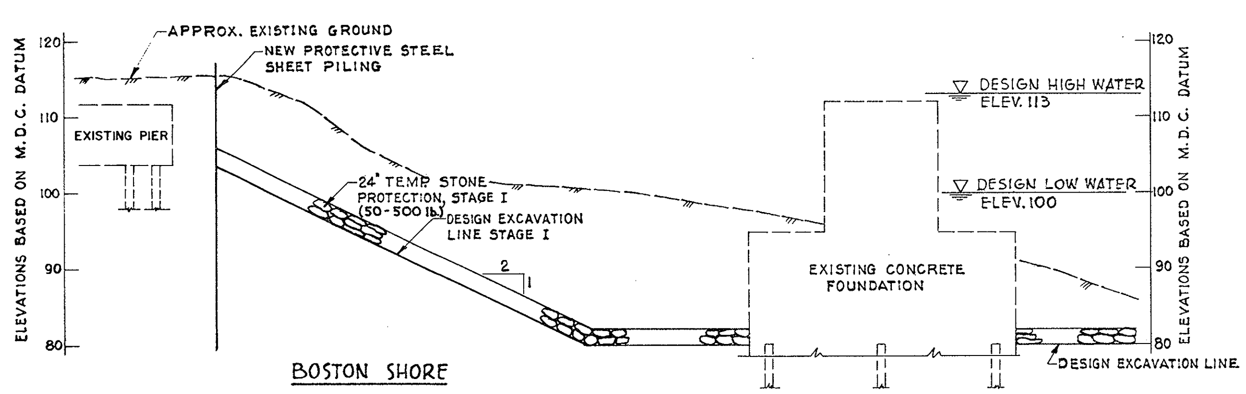
PLATE 4-20



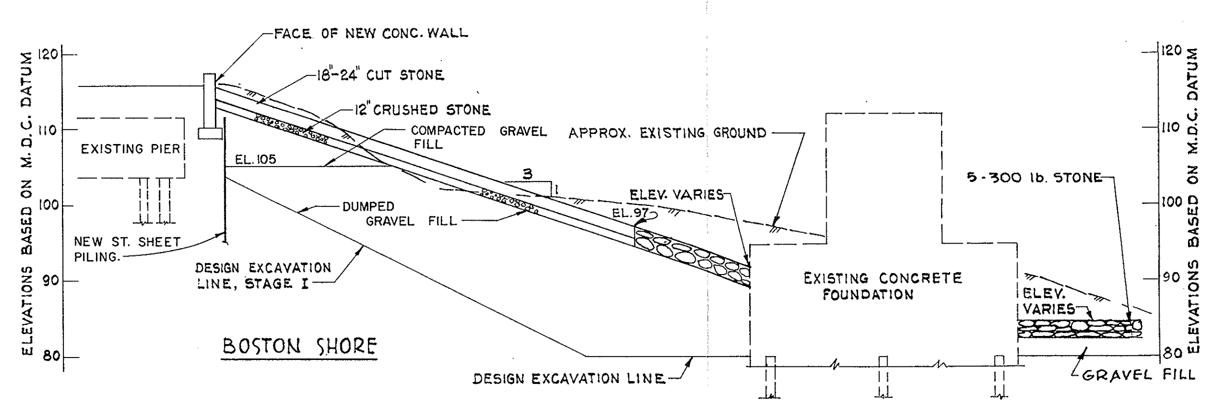
SECTION 3-3



SECTION 7-7



SECTION 8-8, STAGE I

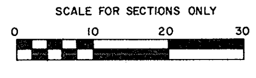


SECTION 8-8, STAGE II

NOTES:

1. Location of Section 3-3 is shown on PLATE 4-16.
2. Locations of Sections 7-7 and 8-8 are shown on PLATE 4-15.

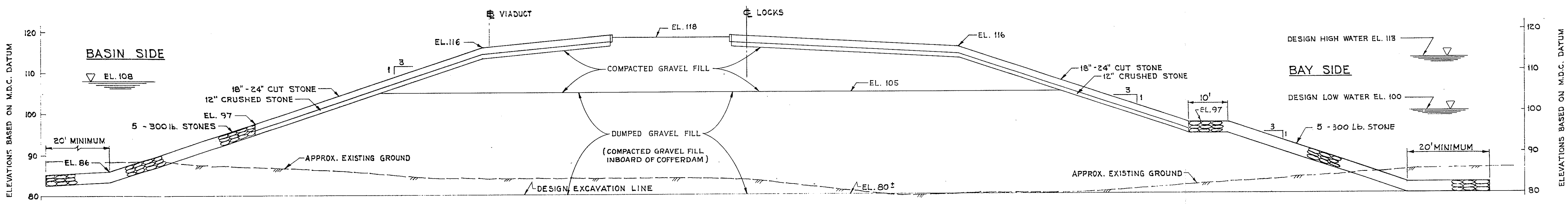
SCALE FOR SECTIONS ONLY



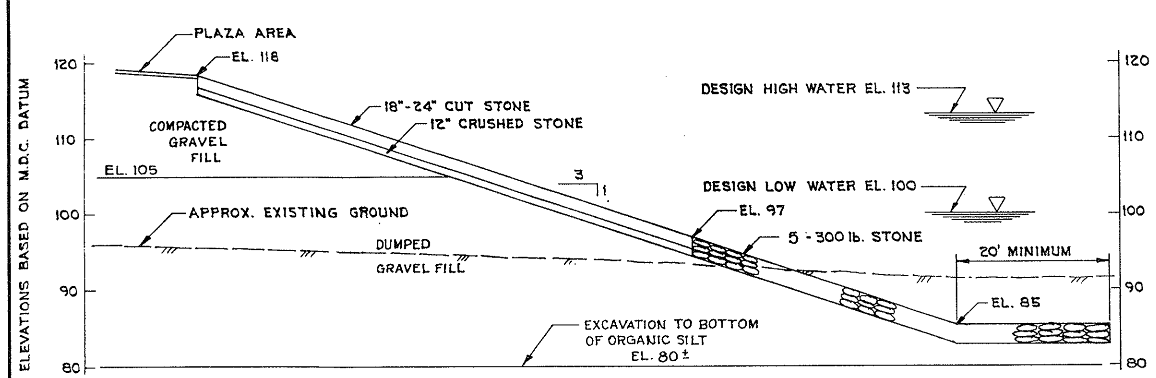
WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
CHARLES RIVER BASIN MASSACHUSETTS

TYPICAL SECTIONS

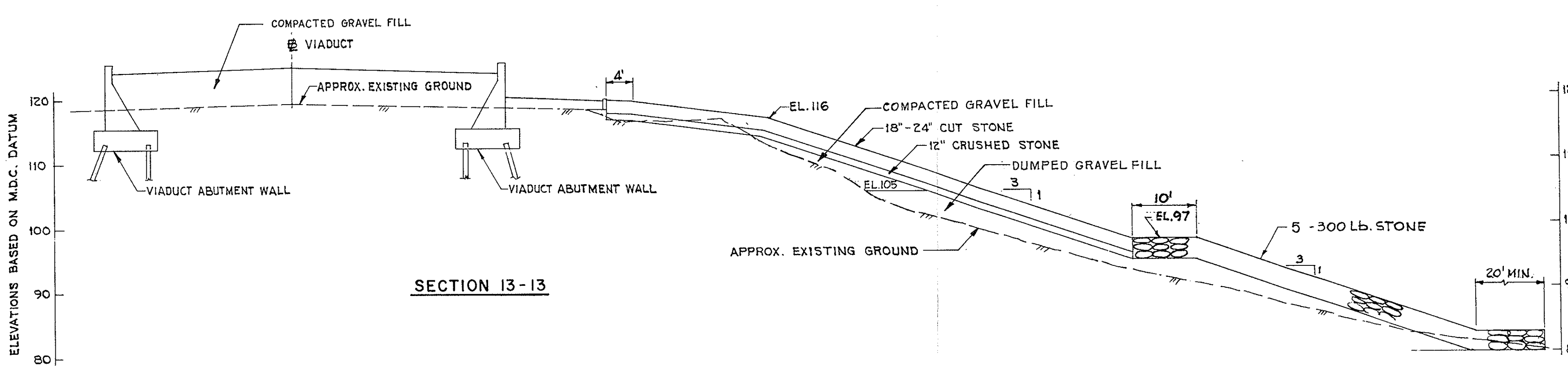
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS WALTHAM, MASS.



SECTION 9-9



SECTION 10-10



SECTION 13-13

NOTES:

1. LOCATIONS OF SECTIONS 9-9 AND 13-13 ARE SHOWN ON PLATE 4-15
2. LOCATION OF SECTION 10-10 IS SHOWN ON PLATE 4-16

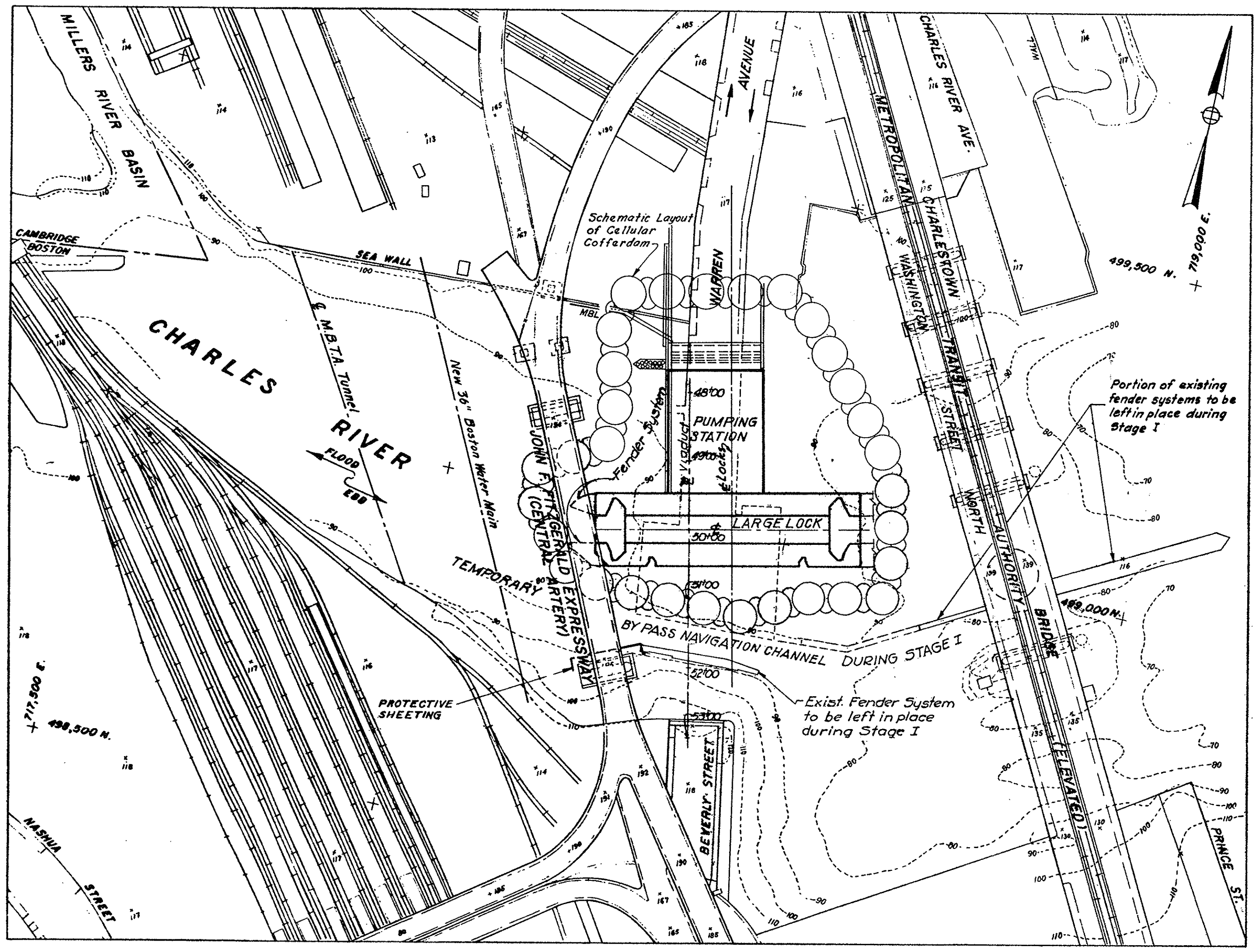
SCALE FOR SECTIONS ONLY
0 10 20 30

GRAPHIC SCALES
1"=20' 20' 0 20' 40'

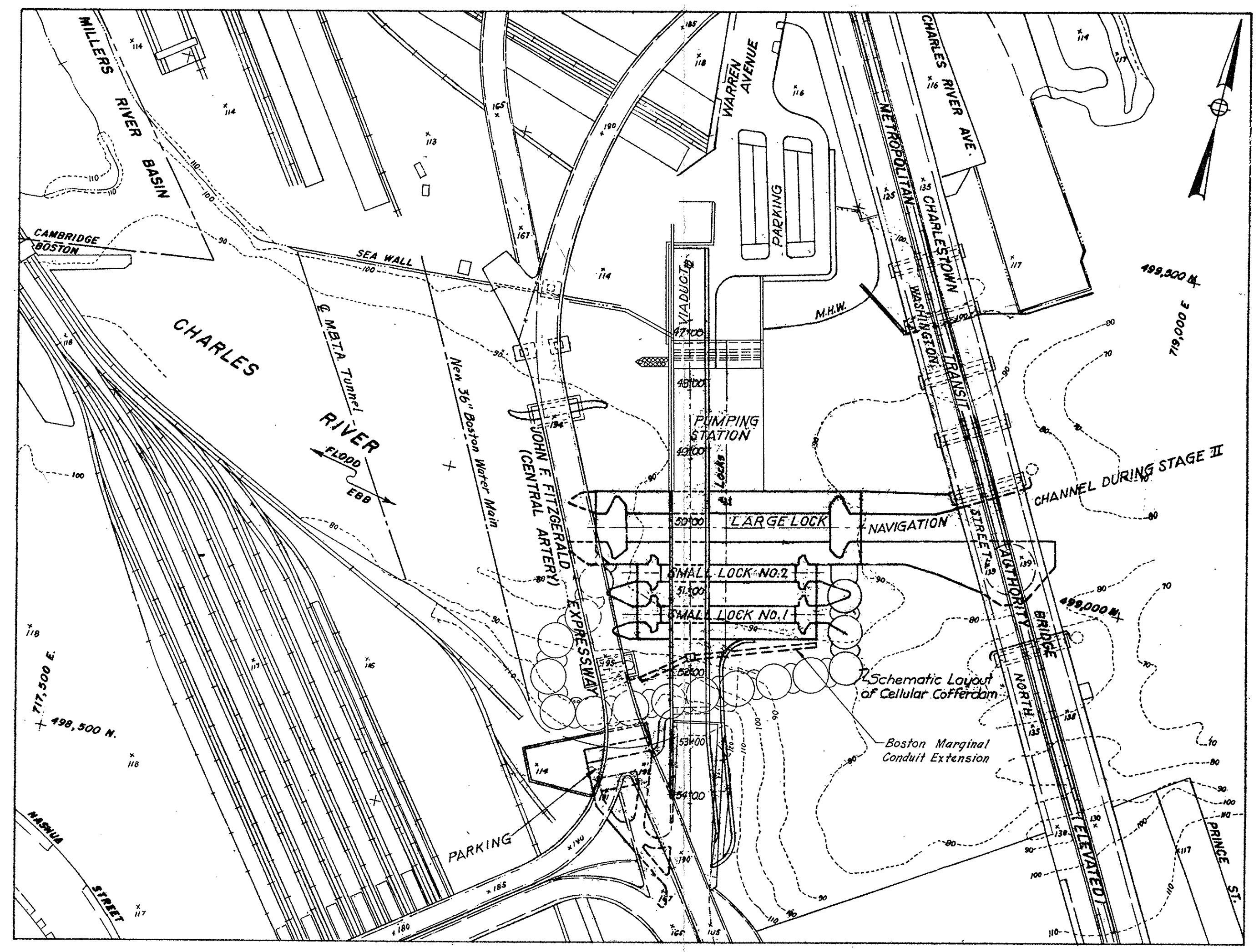
WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM
CHARLES RIVER BASIN MASSACHUSETTS

EMBANKMENT
TYPICAL SECTIONS

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS WALTHAM, MASS.



STAGE I PLAN



STAGE II PLAN

STAGE I CONSTRUCTION:

1. Demolish Existing Facilities.
2. Dredge River Bottom Materials
3. Excavate Temporary By-Pass Navigation Channel and Place Stone Protection.
4. Install Stage I Cofferdam and Protective Sheeting as Required.
5. Dewater Cofferdam Area.
6. Construct Pumping Station, Fish Ladder, Large Lock, Concrete Walls, and Portions of Earth Sections, Utility Systems, Stone Protection and Fender Systems within Stage I Cofferdam.
7. Install Major Equipment and Auxiliaries in Pumping Station.
8. Construct Portions of Viaduct.
9. Complete Stone Protection Up & Downstream of Cofferdam.
10. Test Pumping Station Equipment.
11. Remove Stage I Cofferdam, as required.
12. Complete Earth Sections at Charlestown Side.
13. Test and Place into Operation the Pumping Station and Large Lock, Prior to Stage II Construction.

STAGE II CONSTRUCTION:

1. Excavate and Remove Portion of Temporary Stone Protection.
2. Install Stage II Cofferdam & Dewater.
3. Construct Small Locks, and Portions of Marginal Conduit, Earth Sections, Utility Systems, Stone Protection and Fender Systems within Stage II Cofferdam.
4. Remove Stage II Cofferdam.
5. Complete Earth Sections at Boston Side.
6. Complete Stone Protection.
7. Complete Viaduct.
8. Complete Fender Systems and Other Marine Work.
9. Test and Place into Operation the Small Locks.
10. Complete Landscaping, Roadway and Parking Areas.
11. Final Site Clean-Up.

NOTES:

1. Outline of Stages of Construction operations merely indicate major work to be accomplished and not necessarily in exact sequence of operation. It is presented solely for definition of Stage I and Stage II work to be completed.
2. Contours and Elevations shown are those existing at the time of Field Survey.
3. Layout of Cofferdams is preliminary and subject to change. Final layout will be included in Design Memorandum No. 8, "Cofferdams".

SCALE IN FEET
100 0 100 200 300

REVISION	DATE	DESCRIPTION	BY

CHARLES A. MAGUIRE AND ASSOCIATES INC. WALTHAM, MASSACHUSETTS

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM MASSACHUSETTS

DES. BY | DR. BY | CK. BY

SUBMITTED:
ARCHITECT – ENGINEER
APPROVAL RECOMMENDED:
REVIEWED:
PROJECT ENGINEER
APPROVAL RECOMMENDED:
CHIEF PROJECT BRANCH

WATER RESOURCES DEVELOPMENT PROJECT
CHARLES RIVER LOCKS AND DAM

STAGES OF CONSTRUCTION

CHARLES RIVER BASIN MASSACHUSETTS

APPROVED DATE
CHIEF ENGINEERING DIVISION

SCALE | SPEC. NO.
DRAWING NUMBER
SHEET

APPENDIX A

SUMMARY OF LABORATORY

SOIL TEST RESULTS

APPENDIX A

SUMMARY OF LABORATORY SOIL TESTS RESULTS

PLATE	TITLE
A-1	Soil Tests Results, FD-1, FD-2
A-2	Soil Tests Results, FD-3
A-3	Soil Tests Results, FD-6
A-4	Soil Tests Results, FD-7, FD-8
A-5	Soil Tests Results, FD-9, FD-10
A-6	Soil Tests Results, FD-11, FD-12

SOIL TESTS RESULTS

CHARLES RIVER DAM

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS GRAVEL %	SAND %	FINES %	D_{10} mm.	ATT. LIMITS LL	PL	SPECIFIC GRAVITY	NAT. WATER CONTENT % DRY WT TOTAL	-NO4	COMPACTION DATA STND. AASHO OPT. WATER % DRY WT	MAX. DRY DENS. LBS/CU FT	* PVD LBS/CU FT	NAT. DRY DENSITY LBS/CU FT TOTAL	-NO4	OTHER TESTS SHEAR	CONSOL	PERM.	PERCENT ORGANIC CONTENT
FD-1		J-5	14.4-18.0	OH					79	33	2.60		61									7
		J-12	28.0-31.0	SC	28	36	36	.0017	26	15		9.4	11.4									
		J-13R	28.0-31.0	SC								8.0	9.3									
		J-15	33.0-36.0	SC	25	43	32	.0020	26	15	2.79	12.4	15.5									
FD-1A		UC-1	14.6-16.4	OH	0	7	93	<.001	84	33	2.57	66.5					96.9		Q	C		6
												65.9							R̄			
												60.5										
		UC-3	16.6-18.5	OH	0	6	94	<.001	109	42	2.52	59.0					92.9		Q	C		9
												64.3							R̄			
												71.3										
												73.4										
FD-2		J-5	10.0-15.0	OH	0	8	92		54	28			57									12
		J-10R	20.0-25.0	OH					89	35			65									9
		J-18	40.8-45.0	CL	16	30	54	.001	28	15		9.7	11.2									
		J-19R	40.8-45.0	CL								10.0	11.7									
		J-24	50.0-53.0	CL	10	33	57	.001														

NED FORM JUL 63 510 * PROVIDENCE VIBRATED DENSITY TEST

SOIL TESTS RESULTS

CHARLES RIVER DAM

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	NAT. WATER CONTENT % DRY WT		COMPACTION DATA			NAT. DRY DENSITY LBS/CU FT		OTHER TESTS			
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL		TOTAL	- NO4	STND. AASHO OPT. WATER % DRY WT	STND. AASHO MAX. DRY DENS. LBS/CU FT	* PVD LBS/CU FT	TOTAL	- NO4	SHEAR	CONSOL.	PERM.	
FD-3		J-10R	35.0-39.5	OH									73									
		J-14R	47.0-49.0	SM								20.6	21.4									
		J-21	60.0-64.0	CL	16	26	58	.001	33	17												
		J-22R	60.0-64.0	CL							2.75	10.8	14.8									

NED FORM JUL 63 51C

* PROVIDENCE VIBRATED DENSITY TEST

SOIL TESTS RESULTS

CHARLES RIVER DAM

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS				ATT. LIMITS		SPECIFIC GRAVITY	NAT. WATER CONTENT % DRY WT		COMPACTION DATA			NAT. DRY DENSITY LBS/CU FT		OTHER TESTS		
					GRAVEL %	SAND %	FINES %	D_{10} mm.	LL	PL		TOTAL	-NO4	STND. AASHO OPT. WATER % DRY WT	STND. AASHO MAX. DRY DENS. LBS/CU FT	* PVD LBS/CU FT	TOTAL	-NO4	SHEAR	CONSOL.	PERM.
FD-6		J-4	5.0-11.0	CL					23	15	2.73										
		J-5R	11.0-14.8	CL								10.7	12.5								
		J-6	14.8-15.0	CL	9	36	55	.001													
		J-10R	21.0-24.0	GC								11.0	12.2								
		J-12R	25.0-26.5	GC								8.5	11.4								
		J-14	28.0-30.0	GC					28	16	2.75										
		J-15R	28.0-30.0	GC								9.9	11.5								
		J-18R	31.0-33.2	GC								8.9	11.2								
		J-20R	33.2-34.6	GC								8.2	14.6								

NED FORM JUL 63 510 * PROVIDENCE VIBRATED DENSITY TEST

SOIL TESTS RESULTS

CHARLES RIVER DAM

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS: GRAVEL %	MECHANICAL ANALYSIS: SAND %	MECHANICAL ANALYSIS: FINES %	MECHANICAL ANALYSIS: D_{10} mm.	ATT. LIMITS: LL	ATT. LIMITS: PL	SPECIFIC GRAVITY	NAT. WATER CONTENT % DRY WT: TOTAL	NAT. WATER CONTENT % DRY WT: -NO4	COMPACTION DATA STND. AASHO: OPT. WATER % DRY WT	COMPACTION DATA STND. AASHO: MAX. DRY DENS. LBS/CU FT	COMPACTION DATA: * PVD LBS/CU FT	NAT. DRY DENSITY LBS/CU FT: TOTAL	NAT. DRY DENSITY LBS/CU FT: -NO4	OTHER TESTS: SHEAR	OTHER TESTS: CONSOL.	OTHER TESTS: PERM.	
FD-7		J-6	10.0-12.9	CL					28	16		10.4	11.3									
		J-10	17.0-20.0	SC	20	35	45	.0016	25	15												
		J-11R	17.0-20.0	SC							2.79	8.1	10.9									
		J-12	22.0-24.0	SC					28	17	2.73	9.0	10.8									
		J-15R	33.5-34.9	CL								17.7	18.1									
FD-8		J-3	5.0- 8.0	CL					28	16	2.77											
		J-4R	5.0- 8.0	CL								8.7	10.2									
		J-5	10.0-12.0	CL	14	33	53	.001	28	16												
		J-6R	10.0-12.0	CL								8.9	10.4									
		J-7	22.9-27.0	CL								12.2	13.9									
		J-8R	22.9-27.0	CL	15	23	62	.001			2.78											

NED FORM JUL 63 510 * PROVIDENCE VIBRATED DENSITY TEST

SOIL TESTS RESULTS

CHARLES RIVER DAM

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS: GRAVEL %	MECHANICAL ANALYSIS: SAND %	MECHANICAL ANALYSIS: FINES %	MECHANICAL ANALYSIS: D_{10} mm.	ATT. LIMITS: LL	ATT. LIMITS: PL	SPECIFIC GRAVITY	NAT. WATER CONTENT % DRY WT: TOTAL	NAT. WATER CONTENT % DRY WT: -NO4	COMPACTION DATA STND. AASHO: OPT. WATER % DRY WT	COMPACTION DATA STND. AASHO: MAX. DRY DENS. LBS/CU FT	COMPACTION DATA: * PVD LBS/CU FT	NAT. DRY DENSITY LBS/CU FT: TOTAL	NAT. DRY DENSITY LBS/CU FT: -NO4	OTHER TESTS: SHEAR	OTHER TESTS: CONSOL.	OTHER TESTS: PERM.	
FD-9		J-5	5.0-10.0	SC					30	13	2.79											
		J-6R	5.0-10.0	SC								12.2	13.9									
		J-7	10.0-13.0	GC	40	28	32	.0023														
		J-8R	10.0-13.0	GC								9.4	11.6									
		J-9	16.9-19.0	SC-SM	20	32	48	.0030	21	15												
		J-10R	16.9-19.0	SC-SM							2.74	12.5	14.1									
		J-11	21.8-24.0	SC					25	16	2.78											
		J-12R	21.8-24.0	SC								8.4	9.9									
		J-15R	30.7-32.2	GC								8.3	10.6									
FD-10		J-6	12.0-16.0	SC	22	33	45	.0015	28	16												
		J-7R	12.0-16.0	SC							2.79	12.1	14.2									
		J-10	28.4-31.0	SC-SM	23	39	38	.0035	22	17												
		J-11R	28.4-31.0	SC-SM							2.72											

NED FORM JUL 63 510 * PROVIDENCE VIBRATED DENSITY TEST

SOIL TESTS RESULTS

CHARLES RIVER DAM

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS GRAVEL %	SAND %	FINES %	D_{10} mm.	ATT. LIMITS LL	PL	SPECIFIC GRAVITY	NAT. WATER CONTENT % DRY WT TOTAL	- NO4	COMPACTION DATA STND. AASHO OPT. WATER % DRY WT	MAX. DRY DENS. LBS/CU FT	* PVD LBS/CU FT	NAT. DRY DENSITY LBS/CU FT TOTAL	- NO4	OTHER TESTS SHEAR	CONSOL.	PERM.
FD-11		J-5R	7.0- 9.5	CL								14.3	16.5								
		J-7	10.0-12.9	SC-SM	28	34	38	.0025	22	15											
		J-8R	12.9-15.0	SC-SM							2.78	11.4	13.2								
		J-10R	15.0-15.8	CL								14.4	18.2								
		J-12	21.0-24.0	SC								10.4	13.2								
		J-13	25.0-28.0	SC	21	34	45	.002	24	16	2.78										
FD-12		J-5	16.4-19.0	SM							2.73	6.6	13.2								
		J-6R	16.4-19.0	SM	34	42	24	.0012	18	16											
		J-9R	20.0-25.0	CL					28	17		7.7	11.5								
		J-10	25.0-28.0	SC	14	39	47	.001	25	15											
		J-12R	28.0-31.0	SC								9.3	11.1								

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APPENDIX B

LABORATORY TEST DATA

FOR EXPLORATION FD-1A

APPENDIX B

LABORATORY TEST DATA FOR EXPLORATION FD-1A

PLATE	TITLE
B-1	Undisturbed Sample Log, UC-1
B-2	Gradation Curves, UC-1
B-3	Triaxial Compression (Q) Test Report, UC-1
B-4	Triaxial Compression (R) Test Report, UC-1
B-5	Triaxial Compression (R) Test Report, UC-1
B-6	Undisturbed Sample Log, UC-3
B-7	Gradation Curves, UC-3
B-8	Triaxial Compression (Q) Test Report, UC-3
B-9	Triaxial Compression (R) Test Report, UC-3
B-10	Triaxial Compression (R) Test Report, UC-3

BORING NO. FD-1A
SAMPLE NO. UC-1
DEPTH: 14.60 to 16.43 ft.

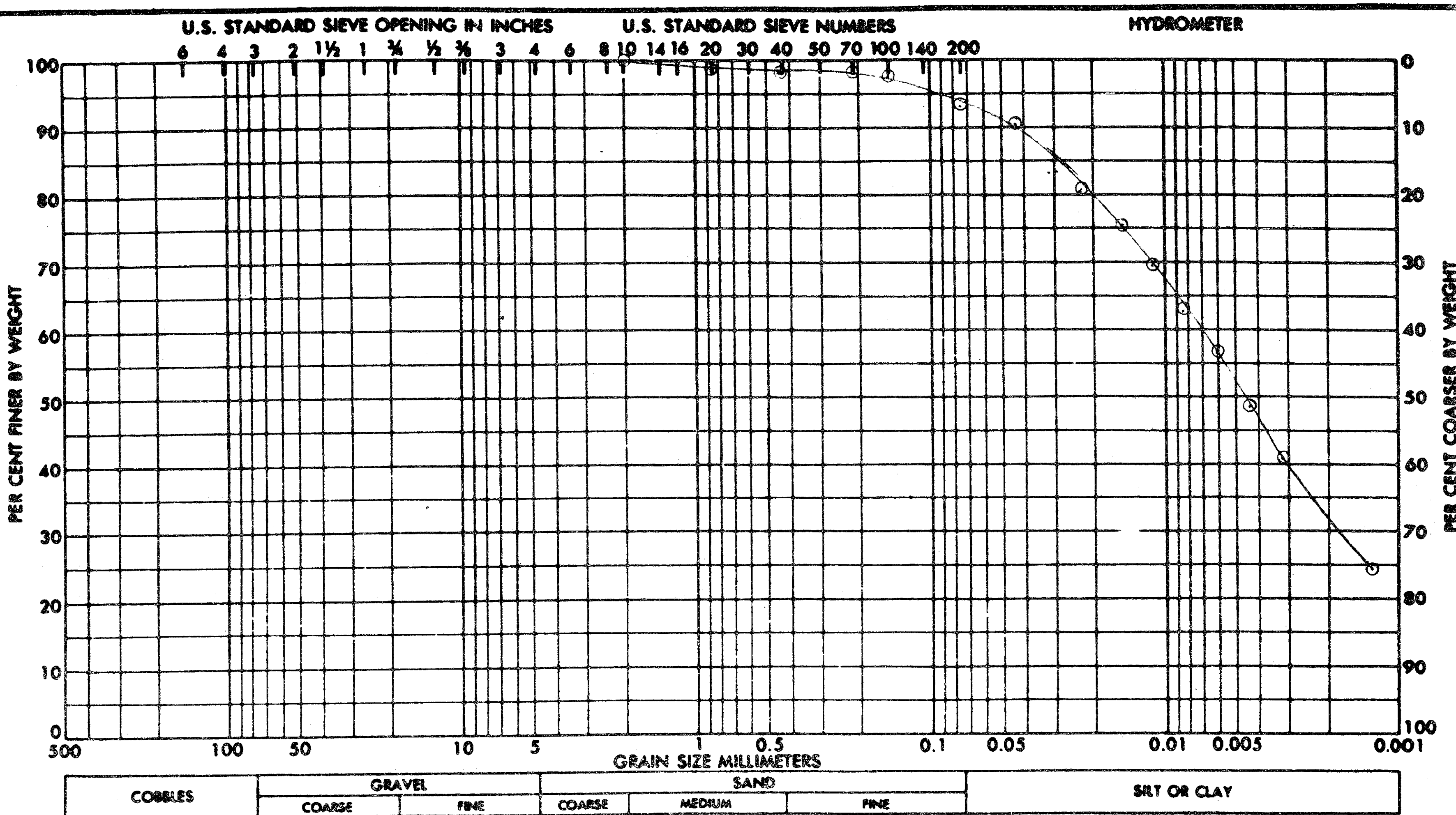
PROJECT Charles River
DATE Feb. 1971
COMP. By J.P.R. CHK'D By R.J.S.

LABORATORY LOG (SAMPLE DEPTH IN FEET)	SAMPLE LENGTH IN INCHES	DESCRIPTION	W, CAN NO.	TEST SAMPLES
	24			
	23			
	22			
	21	Top of sample		
	20			
	19			
	18	Black very moist - wet		
	17	soft organic silty	W_n = 61.8%	R-1, 2, & 3
	16	CLAY (OH) with		
	15	sea shells		
	14			
	13			
	12			
	11			
	10			MA & Hydr.
	9			Org = 5.98%
				Nat. O.D.
	8			LL 84 58
				PL 33 34
	7			PI 51 24
				G = 2.57
	6			
	5		W_n = 66.5%	
	4			Q-1, 2, 3
	3			
	2		W_n = 65.9%	
	1		W_n = 60.5%	C
	0	Bottom of sample		

Length of Sample, L 19.94 in.
Weight of Tube and Wet Soil 17,804 g.
Weight of Tube 7,847 g.
Weight of Wet Soil, W 9,957 g.
Diameter of Tube, D 5.0 in.
Total Unit Weight, $\gamma_t = \frac{4.85\,W}{L D^2}$ 96.9 lbs/cu.ft.

UNDISTURBED SAMPLE LOG

LEGEND
- W_n – Natural Water Content
- MA – Mechanical Analysis
- LL – Atterberg Limits
- G – Specific Gravity
- C – Consolidation
- Q – Unconsolidated Undrained
- γ_d – Dry Density
- R – Consolidated Undrained
- S – Consolidated Drained
- UC – Unconfined Compression



SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
UC-1	14.60'-16.43'	Organic silty CLAY (OH)	Nat	84	33	51
		with sea shells, not included	O.D.	58	34	24
		in gradation				

GRADATION CURVES

PROJECT Charles River

AREA

BORING NO. FD-1A

DATE Feb. 1971

ENG FORM 2087
1 MAY 63

REPLACES WES FORM NO. 1241, SEP 1962, WHICH IS OBSOLETE.

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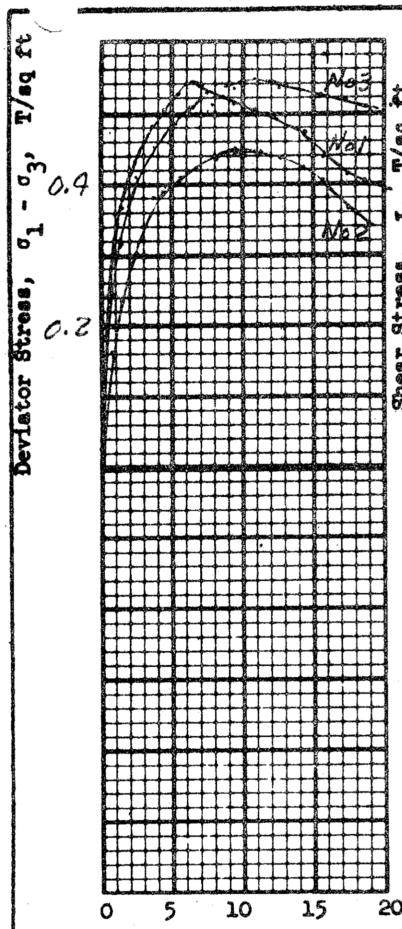
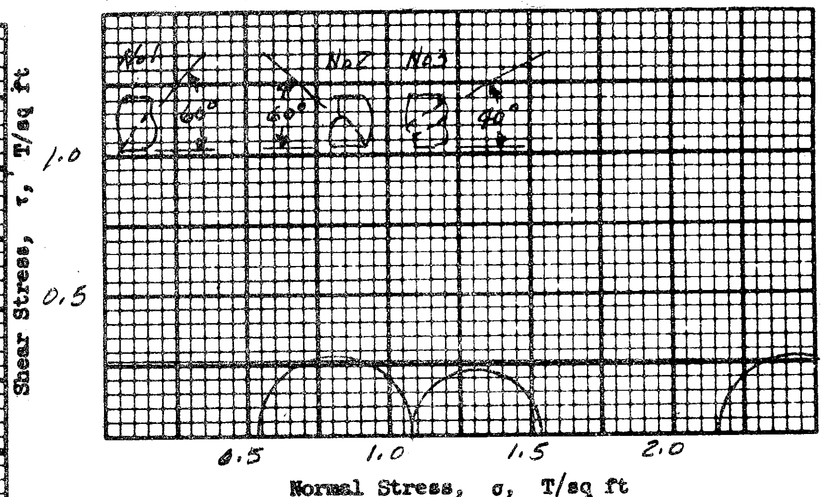




Test No.			1	2	3	
Initial	Water content	w_o	65.3 %	65.9 %	64.6 %	%
	Void ratio	e_o	1.774	1.772	1.786	
	Saturation	S_o	96.2 %	97.5 %	95.7 %	%
	Dry density, lb/cu ft	γ_d	57.9	57.9	57.6	
Before Shear	Water content	w_c	– %	– %	– %	%
	Void ratio	e_c	–	–	–	
	Saturation	S_c	– %	– %	– %	%
	Final back pressure, T/sq ft	u_o	–	–	–	
Final	Water content	w_f	– %	– %	– %	%
	Void ratio	e_f	–	–	–	
Minor principal stress, T/sq ft		σ_3	0.54	1.08	2.16	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	0.54	0.45	0.55	
Time to failure, min		t_f	6.2	9.2	10.8	
Rate of strain, percent/min			1.02	1.02	1.02	
			–	–	–	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$	0.45*	0.41*	0.53*	
Initial diameter, in.		D_o	1.42	1.41	1.42	
Initial height, in.		H_o	3.19	3.16	3.20	

Shear Strength Parameters

ϕ = 0 °

tan ϕ = 0

c = 0.26 T/sq ft

Method of saturation None

☐ Controlled stress

☒ Controlled strain

Type of test Q | Type of specimen Undisturbed

Classification Organic Silty CLAY (OH) with shells

LL	Nat. 84	O.D. 58	PL	Nat. 33	O.D. 34	PI	Nat. 51	O.D. 24	D_{10} <0.001	G_s 2.57

Remarks *Stress @ 15% Strain

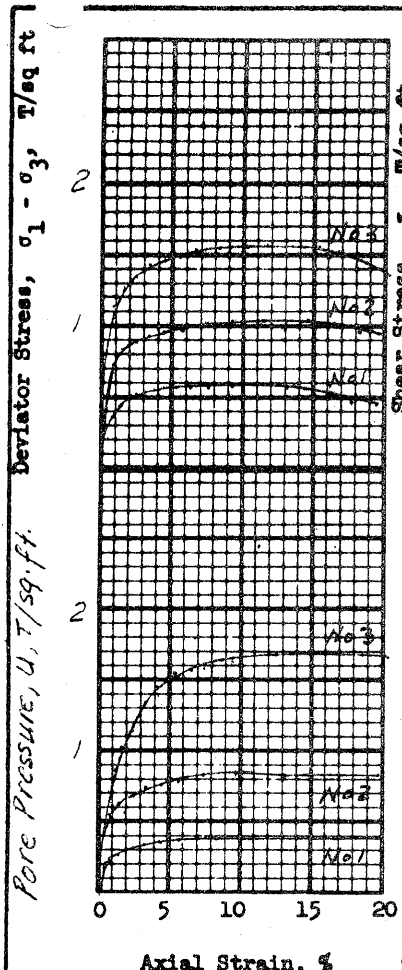
Project Charles River

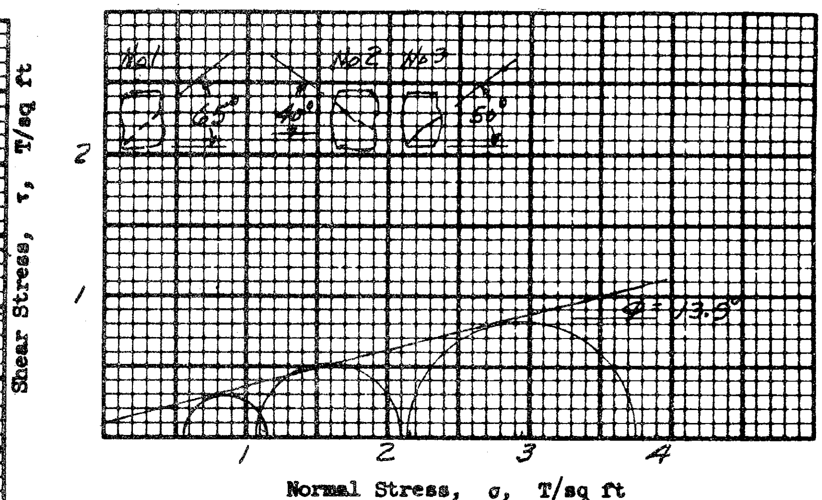
Area

Boring No. FD-1A | Sample No. UC-1

Depth ~~El~~ 14.60' - 16.43' | Date Feb. 1971

TRIAXIAL COMPRESSION TEST REPORT





Test No.			1	2	3	
Initial	Water content	w_o	64.6 %	63.8 %	64.3 %	%
	Void ratio	e_o	1.715	1.667	1.741	
	Saturation	S_o	96.9 %	98.5 %	95.0 %	%
	Dry density, lb/cu ft	γ_d	59.1	60.2	58.6	
Before Shear	Water content	w_c	61.9 %	54.1 %	49.7 %	%
	Void ratio	e_c	1.593	1.392	1.278	
	Saturation	S_c	100 %	100 %	100 %	%
	Final back pressure, T/sq ft	u_o	7.20	7.20	7.20	
Final	Water content	w_f	61.9 %	54.1 %	49.7 %	%
	Void ratio	e_f	1.593	1.392	1.278	
Minor principal stress, T/sq ft		σ_3	0.54	1.08	2.16	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	0.61	1.02	1.58	
Time to failure, min		t_f	83.1	124.6	96.9	
Rate of strain, percent/min			0.12	0.12	0.12	
Pore Pressure T/sq.ft.		U	+0.39	+0.82	+1.67	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$	0.55*	1.02*	1.56*	
Initial diameter, in.		D_o	1.42	1.42	1.41	
Initial height, in.		H_o	3.11	3.13	3.13	

Shear Strength Parameters

ϕ = 13.9 °

tan ϕ = 0.248

c = 0.10 T/sq ft

Method of saturation Backpressure

☐ Controlled stress

☒ Controlled strain

Type of test R	Type of specimen Undisturbed	
Classification Organic silty CLAY (OH) with clam shells		

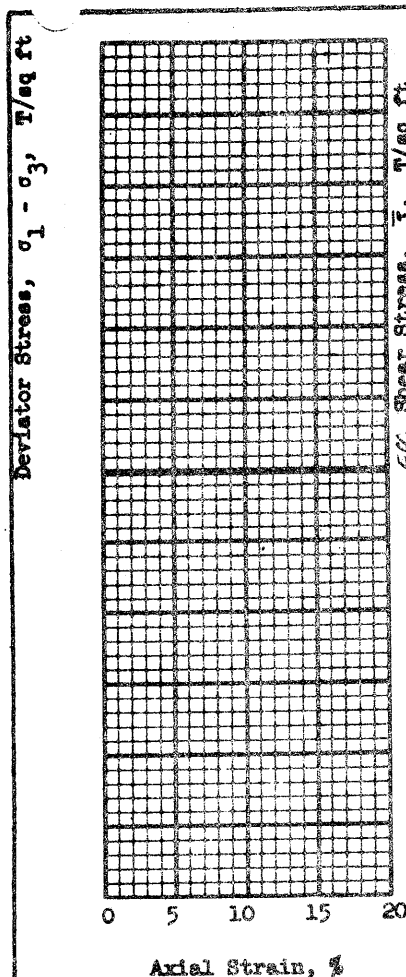
LL	PL	PI		
Nat. 84, O.D. 58	Nat. 33, O.D. 34	Nat. 51, O.D. 24	D_{10} < 0.001	G_s 2.57

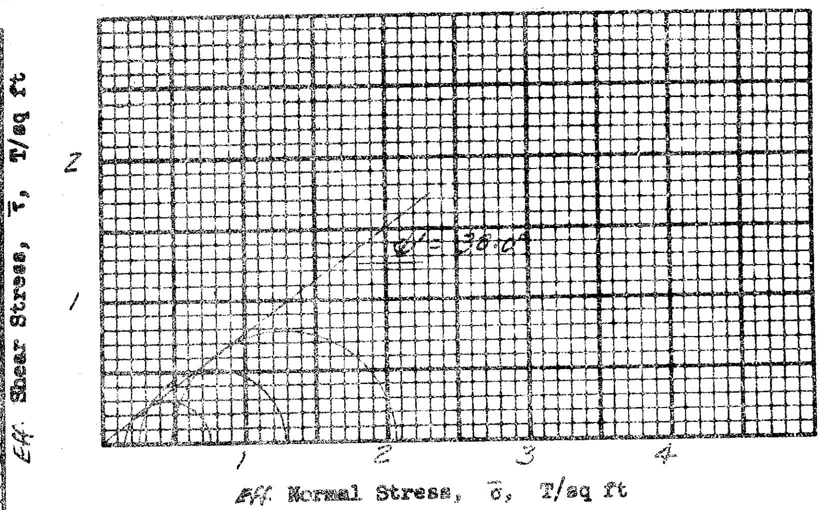
Remarks *Stress @ 15% Strain

Sheet 1 of 2

Project Charles River	
Area	
Boring No. FD-1A	Sample No. UC-1
Depth ~~El~~ 14.60'-16.43'	Date Feb. 1971

TRIAXIAL COMPRESSION TEST REPORT





	Test No.		1	2	3	
Initial	Water content	w_o	%	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d				
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft Eff		$\bar{\sigma}_3$	0.15	0.26	0.49	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$				
Time to failure, min		t_f				
Rate of strain, percent/min						
Major Principal Stress, T/sq.ft. Eff		$\bar{\sigma}_1$	0.75	1.29	2.07	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o				
Initial height, in.		H_o				

Shear Strength Parameters

$\bar{\phi}$ = 38.0°

tan $\bar{\phi}$ = 0.780

$\bar{c}$ = 0 T/sq ft

Method of saturation Backpressure

☐ Controlled stress

☒ Controlled strain

Type of test R	Type of specimen Undisturbed			
Classification Black organic silty CLAY (OH) with clam shells				
LL Nat. 84 O.D. 58	PL Nat. 33 O.D. 34	PI Nat 51 O.D. 24	D_{10} <0.001	G_s 2.57

Remarks See sheet 1 of 2 for other data

Project Charles River	
Area	
Boring No. FD-1A	Sample No. UC-1
Depth El 14.60'-16.43'	Date Feb 1971
TRIAXIAL COMPRESSION TEST REPORT	

Sheet 2 of 2

BORING NO. FD-1A
SAMPLE NO. UC-3
DEPTH: 16.60 to 18.54 ft.

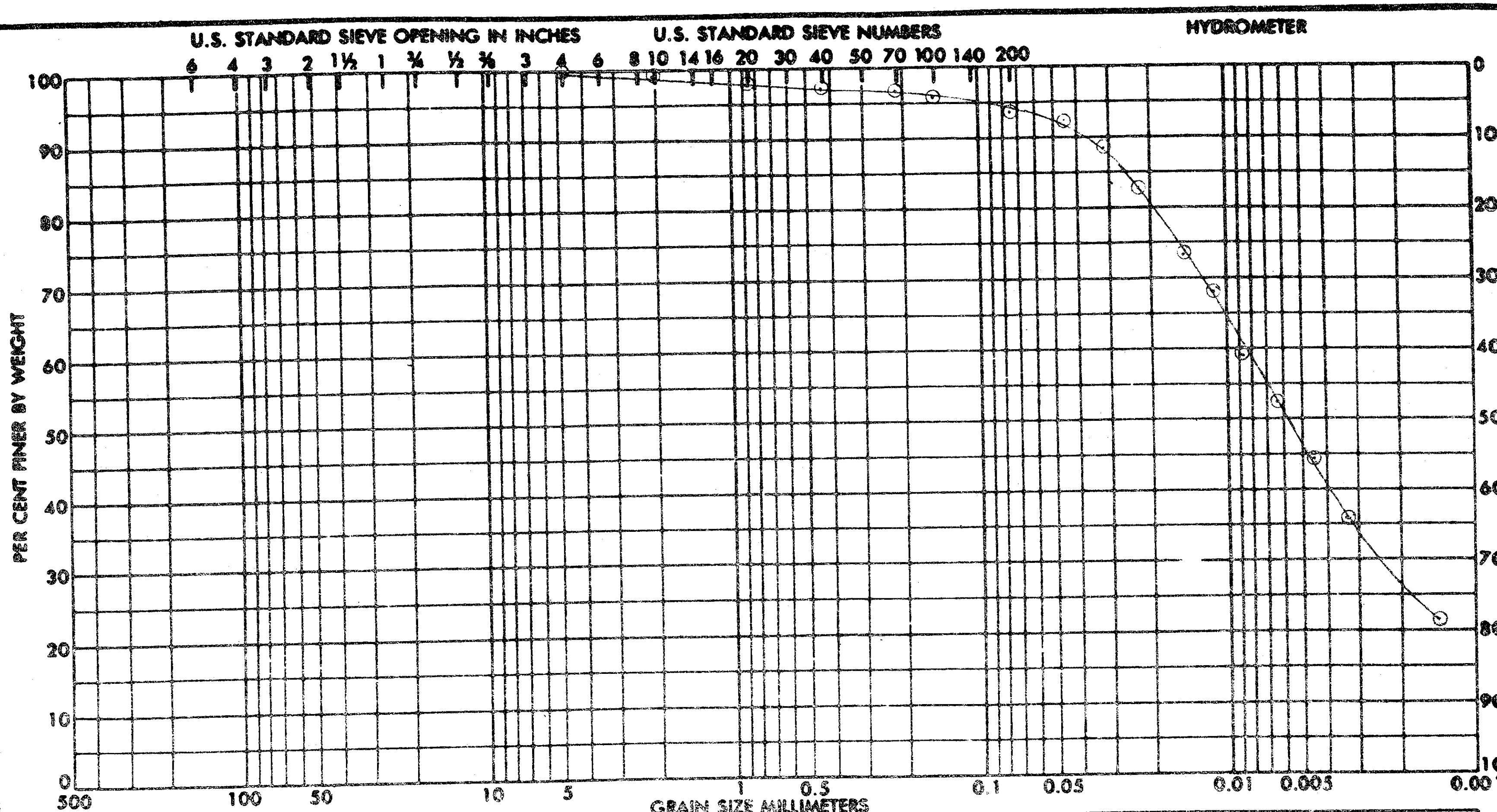
PROJECT Charles River
DATE Feb. 1971
COMP. By JPR CHK'D By RJS

LABORATORY LOG (SAMPLE DEPTH IN FEET)	SAMPLE LENGTH IN INCHES	DESCRIPTION	W, CAN NO.	TEST SAMPLES
	24	Top of Sample		
	23			
	22			
	21	Black, very moist,		
	20	soft organic silty		Q-4
	19	CLAY (OH) with		
	18	numerous clam shells		
	17	and woody fragments		
	16			
	15			
	14			
	13			
	12		Wn = 59.0%	C
	11		Wn = 64.3%	G = 2.52 MA & Hydr
	10			
	9		Wn = 71.3%	R-1, 2, & 3
	8			Nat. O.D. LL 109 72 PL 42 44
	7			PI 67 28
	6		Wn = 73.4%	
	5			Q-1, 2, & 3
	4			Org. = 8.99%
	3			
	2			
	1			
	0	Bottom of Sample		

Length of Sample, L 23.44 in.
Weight of Tube and Wet Soil 19,029 g.
Weight of Tube 7,802 g.
Weight of Wet Soil, W 11,227 g.
Diameter of Tube, D 5.0 in.
Total Unit Weight, $\gamma_t = \frac{4.85\,W}{L D^2}$ 92.9 lbs/cu.ft.

UNDISTURBED SAMPLE LOG

LEGEND
- W_n — Natural Water Content
- MA — Mechanical Analysis
- LL — Atterberg Limits
- G — Specific Gravity
- C — Consolidation
- Q — Unconsolidated Undrained
- γ_d — Dry Density
- R — Consolidated Undrained
- S — Consolidated Drained
- UC — Unconfined Compression



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	NAT W%	LL	PL	PI
UC-5	16.60'-18.54'	Organic silty CLAY (OH)	Nat.	109	42	67
		with 20% clam shells not	O.D.	72	44	28
		included in gradation.				

PROJECT Charles River

AREA

BORING NO. FD-1A

DATE Feb. 1971

GRADATION CURVES

ENG FORM 2087
1 MAY 63

REPLACES WES FORM NO. 1241, SEP 1962, WHICH IS OBSOLETE.

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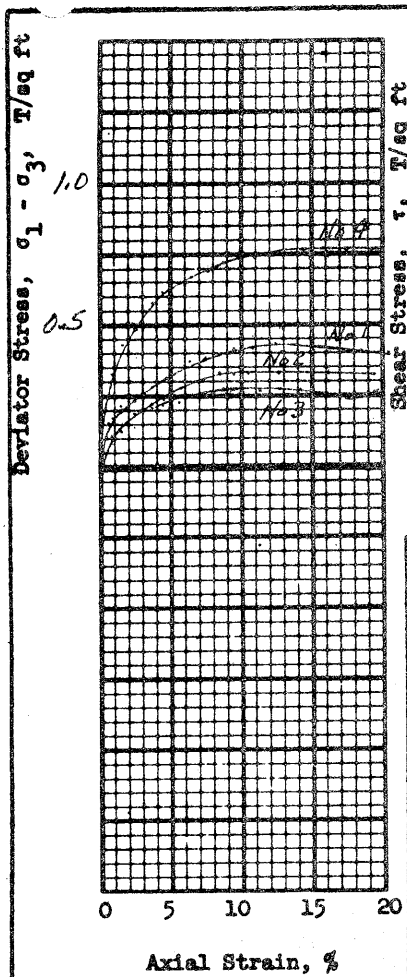
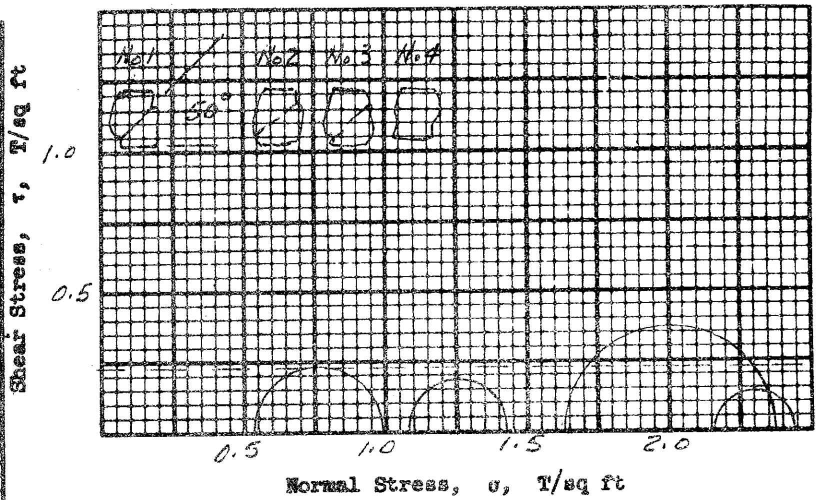
PLATE B-7





Test No.			1	2	3	4
Initial	Water content	w_o	74.6 %	75.4 %	76.7 %	53.3 %
	Void ratio	e_o	2.141	2.091	2.192	1.499
	Saturation	S_o	88.6 %	92.6 %	90.3 %	89.6 %
	Dry density, lb/cu ft	γ_d	50.1	50.9	49.3	62.9
Before Shear	Water content	w_c	— %	— %	— %	— %
	Void ratio	e_c	—	—	—	—
	Saturation	S_c	— %	— %	— %	— %
	Final back pressure, T/sq ft	u_o	—	—	—	—
Final	Water content	w_f	— %	— %	— %	— %
	Void ratio	e_f	—	—	—	—
Minor principal stress, T/sq ft		σ_3	0.54	1.08	2.16	1.62
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	0.44	0.34	0.28	0.76*
Time to failure, min		t_f	12.3	8.0	8.6	14.4
Rate of strain, percent/min			1.01	1.04	1.04	1.04
			—	—	—	—
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$	0.42*	0.33*	0.26*	—
Initial diameter, in.		D_o	1.41	1.41	1.43	1.42
Initial height, in.		H_o	3.23	3.11	3.12	3.13

Shear Strength Parameters

ϕ = 0 °

tan ϕ = 0

c = 0.23 T/sq ft

Method of saturation None

☐ Controlled stress

☒ Controlled strain

Type of test Q | Type of specimen Undisturbed

Classification Organic silty CLAY (OH) with clam shells

LL Nat. 109 O.D. 72 | PL Nat. 42 O.D. 44 | PI Nat. 67 O.D. 28 | D_{10} <0.001 | G_s 2.52

Remarks *Stress @ 15% strain

Project Charles River

Area

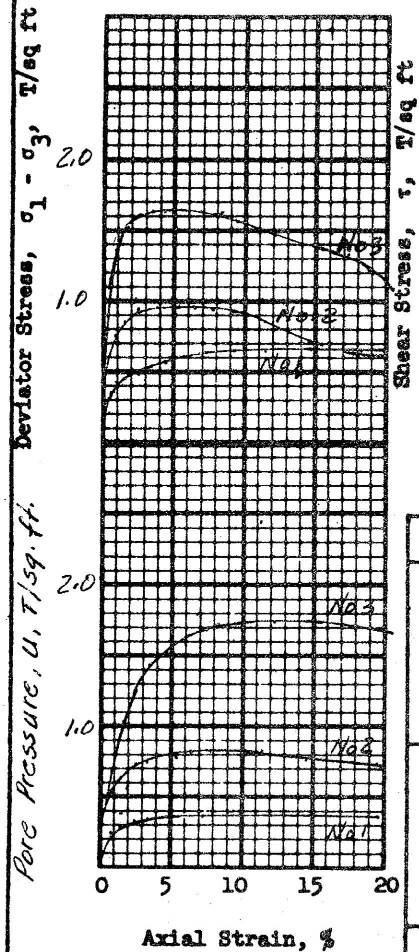
Boring No. FD-1A | Sample No. UC-3

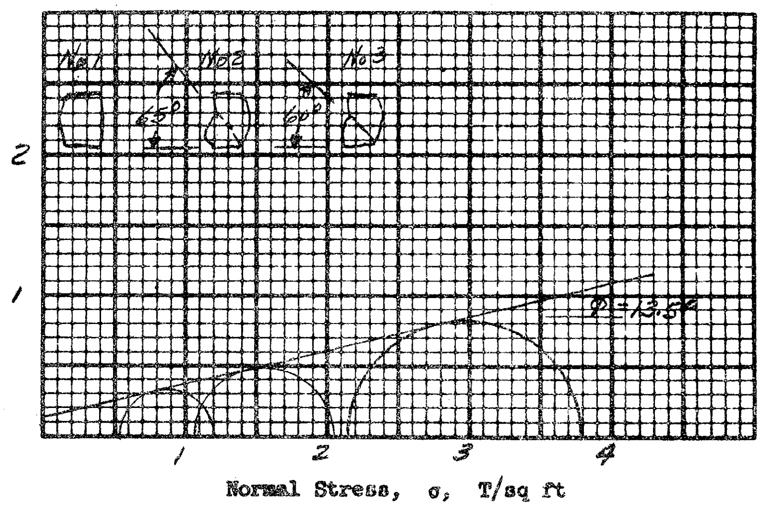
Depth ~~El~~ 16.60'-18.54' | Date Feb. 1971

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 (EM 1110-2-1902) PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT





Shear Strength Parameters

ϕ = 13.5 °

tan ϕ = 0.240

c = 0.14 T/sq ft

Method of saturation Backpressure

☐ Controlled stress

☒ Controlled strain

	Test No.		1	2	3	
Initial	Water content	w_o	75.7 %	80.1 %	74.3 %	%
	Void ratio	e_o	2.148	2.136	2.044	
	Saturation	S_o	88.8 %	94.6 %	91.7 %	%
	Dry density, lb/cu ft	γ_d	50.0	50.2	51.7	
Before Shear	Water content	w_c	72.0 %	68.5 %	58.7 %	%
	Void ratio	e_c	1.815	1.728	1.481	
	Saturation	S_c	100 %	100 %	100 %	%
	Final back pressure, T/sq ft	u_o	7.20	7.20	7.20	
Final	Water content	w_f	72.0 %	68.5 %	58.7 %	%
	Void ratio	e_f	1.815	1.728	1.481	
Minor principal stress, T/sq ft		σ_3	0.54	1.08	2.16	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$	0.67	0.97	1.64	
Time to failure, min		t_f	124.6	45.5	44.3	
Rate of strain, percent/min			0.12	0.11	0.12	
Pore Pressure T/sq.ft.		U	+0.37	+0.79	+1.57	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$	0.67*	0.70*	1.40*	
Initial diameter, in.		D_o	1.41	1.41	1.42	
Initial height, in.		H_o	3.16	3.22	3.12	

Type of test R | Type of specimen Undisturbed

Classification Organic silty CLAY (OH) with Clam Shells

LL Nat 109 O.D. 72 | PL Nat. 42 O.D. 44 | PI Nat. 67 O.D. 28 | D_{10} < 0.001 | G_s 2.52

Remarks *Stress @ 15% strain.

Sheet 1 of 2

Project Charles River

Area

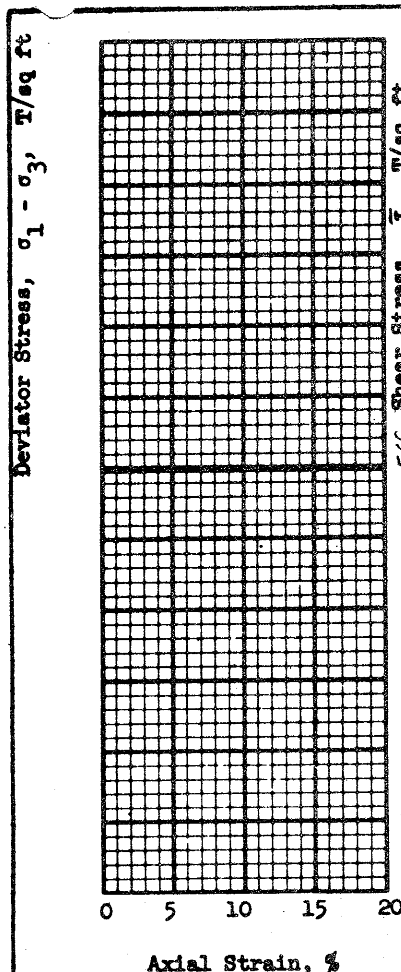
Boring No. FD-1A | Sample No. UC-3

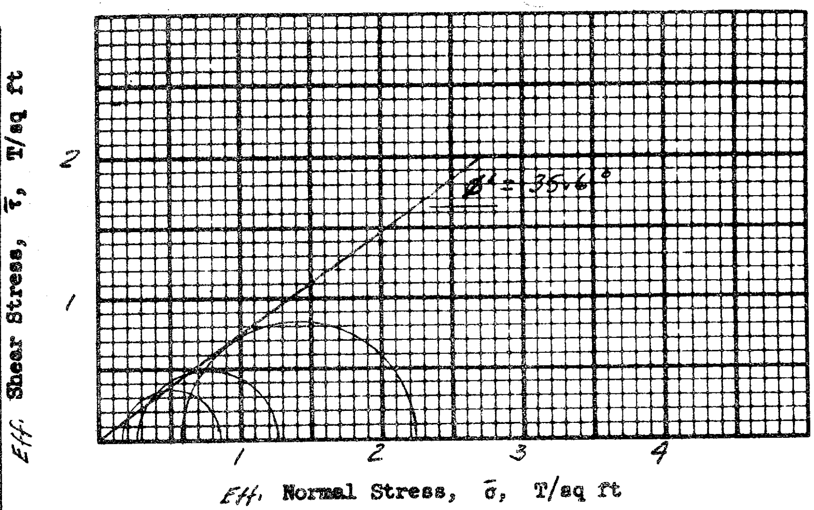
Depth El 16.60' - 18.54' | Date Feb. 1971

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUN 65 2089 (EM 1110-2-1902) PREVIOUS EDITIONS ARE OBSOLETE

TRANSLUCENT





Test No.			1	2	3	
Initial	Water content	w_o	%	%	%	%
	Void ratio	e_o				
	Saturation	S_o	%	%	%	%
	Dry density, lb/cu ft	γ_d				
Before Shear	Water content	w_c	%	%	%	%
	Void ratio	e_c				
	Saturation	S_c	%	%	%	%
	Final back pressure, T/sq ft	u_o				
Final	Water content	w_f	%	%	%	%
	Void ratio	e_f				
Minor principal stress, T/sq ft Eff.		$\bar{\sigma}_3$	0.17	0.29	0.59	
Max deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{max}$				
Time to failure, min		t_f				
Rate of strain, percent/min						
Major Principal Stress, T/sq.ft. Eff.		$\bar{\sigma}_1$	0.84	1.26	2.23	
Ult deviator stress, T/sq ft		$(\sigma_1-\sigma_3)_{ult}$				
Initial diameter, in.		D_o				
Initial height, in.		H_o				

Shear Strength Parameters

$\phi' =$ 35.6 °

$\tan \phi' =$ 0.716

$c' =$ 0 T/sq ft

Method of saturation Backpressure

☐ Controlled stress

☒ Controlled strain

Type of test $\bar{R}$	Type of specimen Undisturbed				
Classification Organic silty CLAY (OH) with Clam Shells					
LL Nat. 109 O.D. 72	PL Nat. 42 O.D. 44	PI Nat 67 O.D. 28	D_{10} <0.001	G_s 2.58	

Remarks

Sheet 2 of 2

Project Charles River	
Area	
Boring No. FD-1A	Sample No. UC-3
Depth El 16.60' - 18.54'	Date Feb. 1971
TRIAXIAL COMPRESSION TEST REPORT	

APPENDIX C

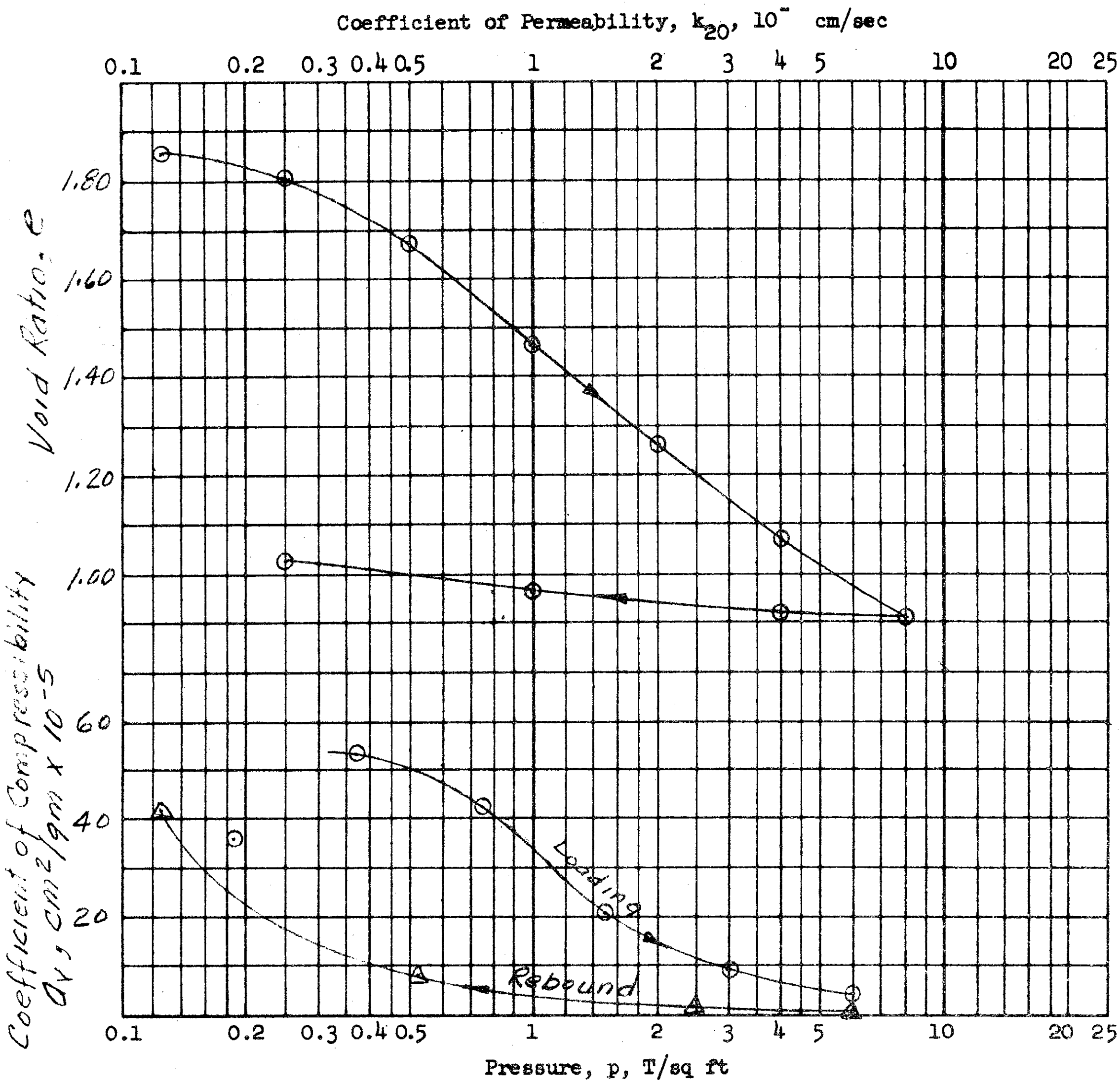
CONSOLIDATION TEST DATA

FOR EXPLORATION FD-1A

APPENDIX C

CONSOLIDATION TEST DATA FOR EXPLORATION FD-1A

PLATE	TITLE
C-1	Consolidation Test Report, UC-1, Sheet 1 of 2
C-2	Consolidation Test Report, UC-1, Sheet 2 of 2
C-3	Consolidation Test Report, UC-3, Sheet 1 of 2
C-4	Consolidation Test Report, UC-3, Sheet 2 of 2



Type of Specimen *Undisturbed*		Before Test	After Test	
Diam *4.44* in.	Ht *1.0* in.	Water Content, w_o *71.2* %	w_f	*43.5* %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o *1.923*	e_f	*1.113*
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o *95.3* %	S_f	*100* %
Compression Index, C_c *.67*		Dry Density, γ_d *54.9* lb/ft^3		
Classification *Organic silty CLAY (OH) with Clam Shells*		k_{20} at e_o = × 10^{-} cm/sec		
LL *Nat. 84* *O.D. 58*	G_s *2.57*	Project *Charles River*		
PL *Nat. 33* *O.D. 34*	D_{10} *<0.001*			
Remarks		Area		
		Boring No. *FD-1A*	Sample No. *UC-1*	
		Depth ~~El~~ *14.60'-16.43'*	Date *March 1971*	
Sheet 1 of 2		**CONSOLIDATION TEST REPORT**		

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE. (TRANSLUCENT)

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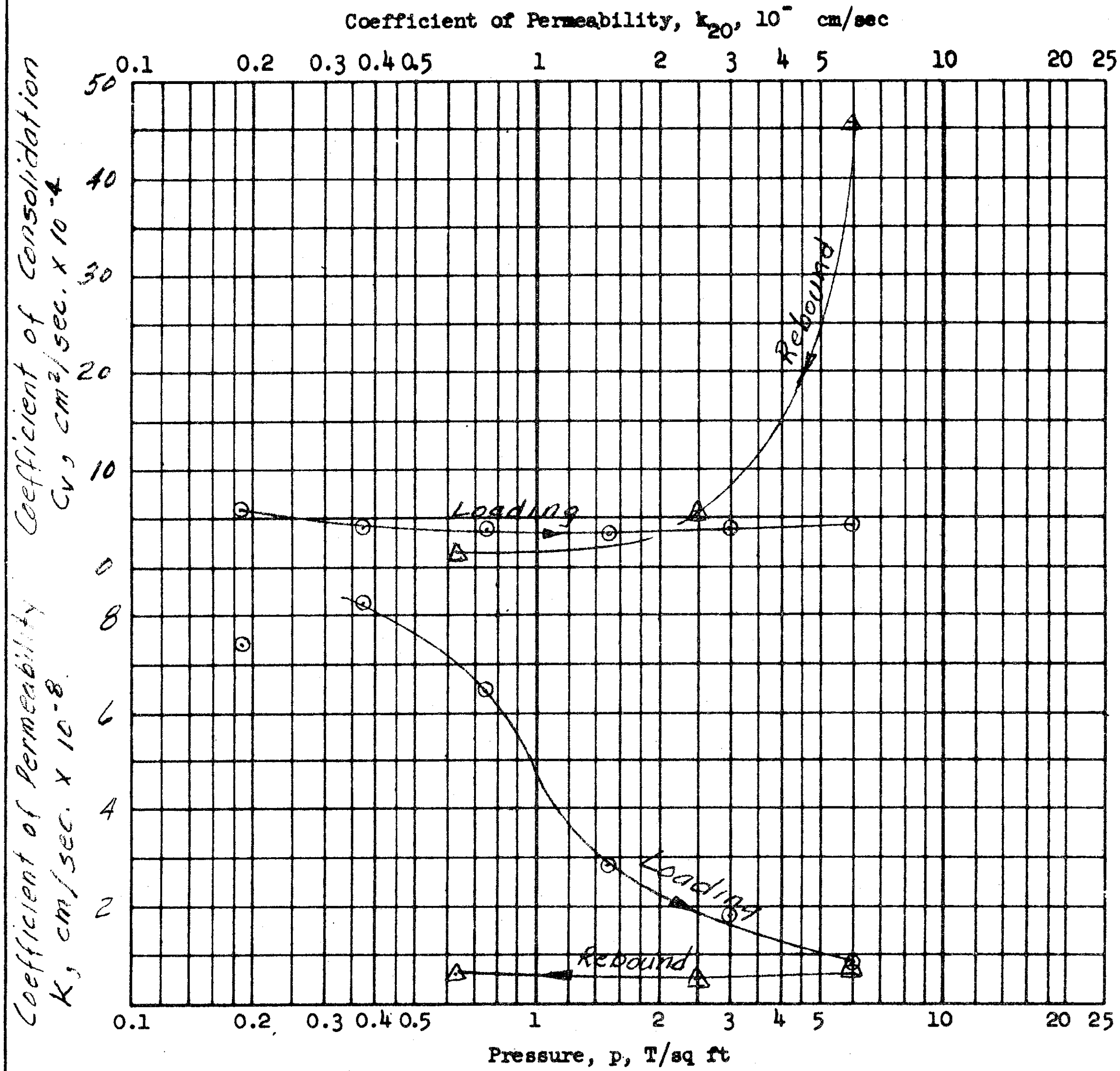
PLATE C-1

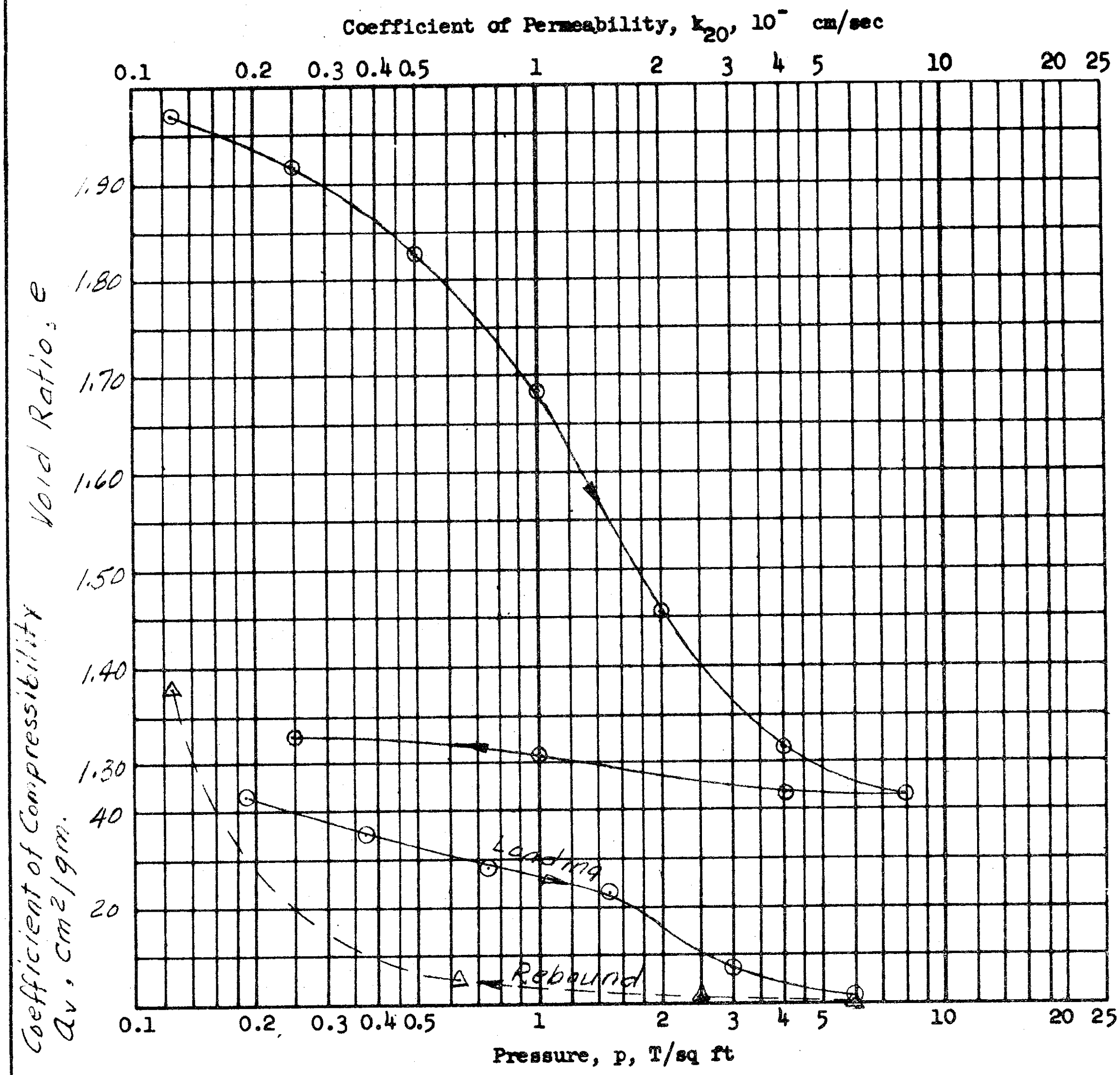


Type of Specimen *Undisturbed*		Before Test		After Test	
Diam *4.44* in.	Ht *1.0* in.	Water Content, w_o	*71.2* %	w_f	*43.5* %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	*1.923*	e_f	*1.113*
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	*95.3* %	S_f	*100* %
Compression Index, C_c		Dry Density, γ_d	*54.9* lb/ft^3		
Classification *Organic Silty CLAY (OH) With Clam Shells*		k_{20} at e_o = × 10^{-} cm/sec			

LL *Nat. 84* *O.D. 58*	G_s *2.57*	Project *Charles River*
PL *Nat. 33* *O.D. 34*	D_{10} *<0.001*	
Remarks *Permeability values computed from consolidation data.*		Area
		Boring No. *FD-1A* — Sample No. *UC-1*
		Depth El *14.60' - 16.43'* — Date *March 1971*
Sheet 2 of 2		**CONSOLIDATION TEST REPORT**

ENG FORM 2090 1 MAY 63 PREVIOUS EDITIONS ARE OBSOLETE.

PLATE C-2



Type of Specimen *Undisturbed*		Before Test		After Test	
Diam *4.44* in.	Ht *1.0* in.	Water Content, w_o	*72.0* %	w_f	*54.5* %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	*2.046*	e_f	*1.501*
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	*88.8* %	S_f	*91.6* %
Compression Index, C_c		Dry Density, γ_d	*51.7* lb/ft³		
Classification *Organic silty CLAY (OH) w/clam shells*		k_{20} at e_o = × 10⁻ cm/sec			

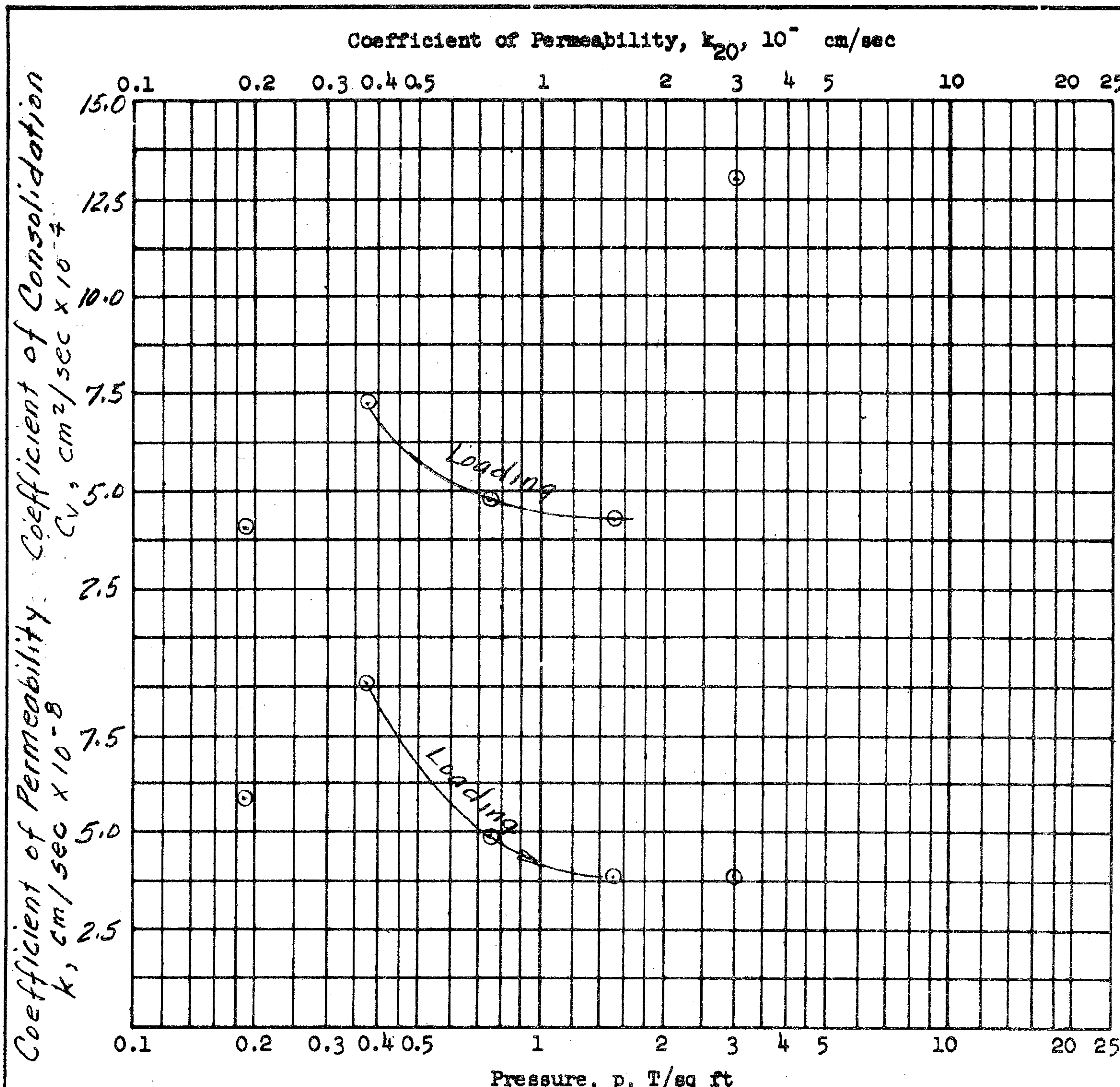
LL Nat. *109* O.D. *72*	G_s *2.52*	Project *Charles River*
PL Nat. *42* O.D. *44*	D_{10} *<0.001*	

Remarks *Large amount of clam shells in sample*

Area	
Boring No. *FD-1A*	Sample No. *UC-3*
Depth ~~El~~ *16.60'-18.54'*	Date *March 1971*

CONSOLIDATION TEST REPORT

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE.



Type of Specimen *Undisturbed*		Before Test		After Test	
Diam *4.44* in.	Ht *1.0* in.	Water Content, w_o	*72.0* %	w_f	*54.5* %
Overburden Pressure, p_o	T/sq ft	Void Ratio, e_o	*2.046*	e_f	*1.501*
Preconsol. Pressure, p_c	T/sq ft	Saturation, S_o	*88.8* %	S_f	*91.6* %
Compression Index, C_c		Dry Density, γ_d	*51.7* lb/ft³		
Classification *Organic Silty CLAY (OH) W/ clam shells*		k_{20} at e_o =	× 10⁻ cm/sec		

LL Nat *109* O.D *72*	G_s *2.52*	Project *Charles River*
PL Nat *42* O.D. *44*	D_{10} *< 0.001*	
Remarks *Permeability computed from consolidation data. Large amount of clamshells in sample.*		Area
		Boring No. *FD-1A* — Sample No. *UC-3*
		Depth ~~El~~ *16.60'-18.54'* — Date *March 1971*
		CONSOLIDATION TEST REPORT

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE.